U.S. DEPARTMENT OF COMMERCE
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PB-247 163

SHORT COURSE PROCEEDINGS

APPLICATIONS OF STORMWATER MANAGEMENT MODELS

MASSACHUSETTS UNIVERSITY

PREPARED FOR
NATIONAL ENVIRONMENTAL RESEARCH CENTER

JUNE 1975

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EPA-670/2-75-065
June 1975

Environmental Protection Technology Series

UNITED STATES · ENVIRONMENTAL PROTECTION AGENCY

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Springfield, VA. 22151

TECHNICAL REPORT DATA

(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-670/2-75-065	2.	3. RECIPIENT'S ACCESSION NO. PB247163

4. TITLE AND SUBTITLE Short Course Proceedings: APPLICATIONS OF STORMWATER MANAGEMENT MODELS	5. REPORT DATE June 1975 (Issuing Date) 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Editors: Francis A. DiGiano and Peter A. Mangarella	8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Department of Civil Engineering University of Massachusetts Amherst, Massachusetts 01002	10. PROGRAM ELEMENT NO. 1BB034 (ROAP 21-ATA, Task 026) 11. CONTRACT/GRANT NO. R-803069
12. SPONSORING AGENCY NAME AND ADDRESS National Environmental Research Center Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268	13. TYPE OF REPORT AND PERIOD COVERED 1974 Short Course Proceedings 14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

16. ABSTRACT

This Short Course was sponsored by the U.S. Environmental Protection Agency. The specific objectives were to encourage the consulting profession to implement stormwater models in solving the problem of storm and combined sewer overflows and to make state pollution control agencies aware of this tool in their pollution abatement efforts. Emphasis was placed on presentations of various types of models, their data requirements and case studies of their use. The EPA Stormwater Management Model (SWMM) was highlighted. It is hoped that this compilation of instructional papers, prepared by the Short Course faculty, will enable practicing engineers to broaden their use of stormwater management models. The Short Course was held at the University of Massachusetts August 19-23, 1974. Registration totaled 81 with representation by consultants; Federal, State and Municipal engineers, including the Canadian government; and University researchers. This report submitted in partial fulfillment of Project Number R-803069-01-1 by the Department of Civil Engineering at the University of Massachusetts, under the sponsorship of the Environmental Protection Agency.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Computer programs, *Mathematical models, *Storm sewers, *Combined sewers, *Simulation, *Rainfall intensity, *Runoff, Pollution, *Stream pollution, *Waste treatment, Cost analysis, Cost effectiveness, Optimization, *Water quality	*Infiltration, *Peak storm flow, Combined sewer overflows, Urban runoff, Water quality control	13B

PRICES SUBJECT TO CHANGE

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 436
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE 11.75 - 2.25

EPA Form 2220-1 (9-73)

☆ U.S. GOVERNMENT PRINTING OFFICE: 1975 657 594/5410 Region No. 5-11

EPA-670/2-75-065
June 1975

Short Course Proceedings

APPLICATIONS OF STORMWATER MANAGEMENT MODELS

August 19-23, 1974
University of Massachusetts/Amherst

Edited By

Francis A. DiGiano
Peter A. Mangarella
University of Massachusetts
Amherst, Massachusetts 01002

Project No. 803069
Program Element No. 1BB034

Project Officer

Chi-Yuan Fan
Storm and Combined Sewer Section (Edison, N.J.)
Advanced Waste Treatment Research Laboratory
National Environmental Research Center
Cincinnati, Ohio 45268

NATIONAL ENVIRONMENTAL RESEARCH CENTER
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

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FOREWORD

Man and his environment must be protected from the adverse effects of pesticides, radiation, noise and other waste. Efforts to protect the environment require a focus that recognizes the interplay between the components of our physical environment--air, water and land. The National Environmental Research Centers provide this multidisciplinary focus through programs engaged in

- studies on the effects of environmental contaminants on man and the biosphere, and
- a search for ways to prevent contamination and to recycle valuable resources.

This five day short course was designed for engineers and technical personnel associated with consulting, regulatory, governmental, and educational organizations with an interest and responsibility in the control of storm and combined sewer overflows. The Short Course format provided for lecture and workshop sessions on the development and use of computer assisted models (with special emphasis on the EPA Stormwater Management Model) for the prediction and control of urban stormwater quantity and quality.

A. W. Breidenbach, Ph.D.
Director
National Environmental
Research Center, Cincinnati

ABSTRACT

This Short Course was sponsored by the U. S. Environmental Protection Agency. The specific objectives were to encourage the consulting profession to implement stormwater models in solving the problem of storm and combined sewer overflows and to make state pollution control agencies aware of this tool in their pollution abatement efforts. Emphasis was placed on presentations of various types of models, their data requirements and case studies of their use. The EPA Stormwater Management Model (SWMM) was highlighted. It is hoped that this compilation of instructional papers, prepared by the Short Course faculty, will enable practicing engineers to broaden their use of stormwater management models.

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ACKNOWLEDGEMENTS

We would especially like to thank Mr. Richard Field, Chief and Mr. Chi-Yuan Fan, Project Officer of the Storm and Combined Sewer Section and Mr. Harry Torno, Staff Engineer of the Municipal Pollution Control Division of the EPA for their cooperation throughout the planning and execution stages of this Short Course. Their help in establishing the program and topics of the Short Course has been of immeasurable value.

We would also like to thank those speakers who participated in this Short Course. Special recognition is to be given to Mr. Thomas K. Jewell, graduate research assistant, and Mr. David Gaboury, undergraduate research assistant, for their tireless effort in applying the EPA SWMM to the UMASS computer system and in developing a case study for Greenfield, Massachusetts.

We are also grateful to the staff of the Continuing Education Conference Management Center for handling all of the Short Course details. Finally, we acknowledge the help of our Environmental Engineering secretarial staff, Miss Dorothy A. Blasko and Ms. Phyllis L. Mayberg, for their help in proofing the lecture materials.

Francis A. DiGiano
Short Course Co-Chairman

Peter A. Mangarella
Short Course Co-Chairman
University of Massachusetts/Amherst

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SHORT COURSE FACULTY

Bernard B. Berger, Director
Water Resources Research Center
University of Massachusetts

Albin Brandstetter, Ph.D.
Environmental Resources Planning
Section
Battelle Pacific Northwest Laboratories
Richland, Washington

Francis A. DiGiano, Ph.D.
Short Course Co-Chairman
Assistant Professor of Civil Engineering
University of Massachusetts

Richard Field, Chief
Storm & Combined Sewer Section
U. S. Environmental Protection Agency

James P. Heaney, Ph.D.
Associated Professor of
Environmental Engineering Sciences
University of Florida

Wayne C. Huber, Ph.D.
Associate Professor of
Environmental Engineering Sciences
University of Florida

Thomas K. Jewell
Graduate Research Assistant
Department of Civil Engineering
University of Massachusetts

Peter A. Mangarella, Ph.D.
Short Course Co-Chairman
Assistant Professor of Civil Engineering
University of Massachusetts

Murray B. McPherson, Director
ASCE Urban Water Resources Program

Gerald T. Orlob, Ph.D
Senior Partner
G. T. Orlob & Associates
Orinda, California

Larry A. Roesner, Ph.D.
Principal Engineer
Water Resources Enginners, Inc.
Walnut Creek, California

Robert P. Shubinski, Ph.D.
Vice President
Water Resources Engineers, Inc.
Springfield, Virginia

Jekabs P. Vittands, Project Mgr.
Metcalf & Eddy Consulting
Engineers
Boston, Massachusetts

David E. Winslow, Manager
Espey-Huston and Associates, Inc.
Houston, Texas

A GENERAL COMMENT ON STORMWATER MANAGEMENT MODELS

By

Bernard B. Berger*

We shall examine this week two sets of procedures concerned with urban storm flow. I refer first to procedures for predicting the quantity of urban runoff, given data on frequency, intensity and duration of storms traversing the urban area; and, second, to procedures for predicting the mass of specific pollutants conveyed by the resulting runoff to points of disposal. The first set, with key word "quantity", represents a general methodology that has long been familiar to the civil engineer-hydrologist. The second, with key word "quality", represents a technique that is still experimental and uncertain reflecting as it does (1) community values and habits in terms of unrecorded and often undetected wastes in addition to street debris and dirt; (2) transport properties of sheets of surface water in terms of pollutant pickup and conveyance; and, (3) mass transport of pollutants contained in mixtures of sewage and storm runoff that constitute combined sewer overflow. Management of "quality" requires in some degree management of the storm flow.

Were we interested in flow per se, our effort would obviously be directed only to flood prediction and control - historically and traditionally, a major area of civil engineering concern. But our interest is, of course, not so limited. We proceed under a compulsion to cleanse our streams and keep them clean.

The problem of water quality deterioration produced by storm runoff is widely experienced, and it is severe. It applies to rural areas and to urban areas, to lands developed by man and to undeveloped lands. We recognize, however, that the urban area presents a special case because its impervious surfaces and its conduits increase, quicken, and concentrate storm flow, and because such flow encounters and picks up a variety of substances that may be grossly pollutional.

---

*Director, Water Resources Research Center, University of Massachusetts/ Amherst.

As engineers and environmentalists we seek methods for protecting our water resources against such damaging substances. In so doing we are bound to ask such questions as:

- What is the nature of these materials and where and how do they originate?
- How do their effects vary with frequency, intensity and duration of storms?
- When is it desirable to capture storm runoff so that controls may be applied to protect the quality of receiving waters?
- What types of controls are feasible and what conditions apply to their use?
- To what extent may such materials be controlled through improved programs of community cleanliness, and through land use regulation and zoning?
- When is a sewer separation program a reasonable solution for the combined sewer overflow problem?

These and similar questions are of compelling importance in view of the goals of the Water Quality Act of 1972 and its requirements for highly effective end-of-pipe waste treatment technology. However, answers to these questions are useful only if such information can be applied dependably to predictions on the resulting quality of the receiving water. It is, therefore, appropriate that we examine cause and effect relationships in this context of water quality management. In doing this we must also ask: can we with confidence, determine the magnitude of improvement in water quality that would result from a specific investment in control of an urban non-point source of pollution? Is it possible to design a program of control that is optimally balanced between economic and environmental consideration?

We may ask many additional questions, all of them important. To answer them we must have available a predictive system which encompasses all significant variables; fitting and weighting them in a way that provides a reliable picture of consequences of an intricate set of pollutional stresses and correctives. The development of such a predictive model would appear to be as challenging an assignment as any yet faced by the civil-environmental engineering profession. Such systems have, in fact, been proposed. We shall examine these systems during this week's short course.

It is surprising to me that we have been so slow in recognizing the need for a reliable storm water quality prediction and management model. Perhaps the reason lay in the assumption that pollution by storm waters hardly seemed important in view of existing, extensive discharges of raw sewage and primary plant effluents. But the fact is we were slow to comprehend the full extent of pollution by combined sewer overflows. This was so in the face of our knowledge that major interceptors installed three-quarters of a century or longer ago were fixed in carrying capacity while the contributing areas and population expanded enormously. We have also been slow in recognizing the limitation of separate storm sewers in protecting the quality of receiving waters.

The frequency of combined sewer overflows came to our attention quite late, actually in 1947 when McKee reported the results of his analysis of Boston's sewerage system in the Journal of the Boston Society of Civil Engineers. According to McKee, overflows occurred 5-7 times monthly or whenever an intensity of precipitation greater than 0.03" per hour was experienced, each increment of 0.01" per hour being equivalent to the normal dry weather flow. This finding was important, of course, and it stimulated general interest. But even so, the connection between overflow and its pollutional impact appears not to have been fully recognized at that time.

This association was made a few years later when studies in Detroit showed that large quantities of organics deposited in the sewer at times of dry weather flow were scoured and discharged in overflows during periods of storm. The Detroit study was found to reflect conditions in many other cities served by combined sewer systems. In 1961, Thomas Camp summarized the situation prevailing in most U.S. cities as follows: "Overflows of mixed sewer and storm water occur nearly every time it rains, or about 5 to 7 times per month on the average. But in those 5 to 7 times per month, as much as 1/3 of the sewage solids produced by the tributary population over a year's time is discharged without treatment into the receiving waters, carrying large numbers of intestinal bacteria and viruses while discharging only about 3% of a year's production of liquid sanitary sewage. The reason for this apparent anomaly is that the flow capacity of combined sewers ranges from about 50 to 200 times the average flow of sanitary sewage. So during dry weather the velocity is so small that sewage solids settle in the sewer and very small rain storms will suspend these materials."

Perhaps I belabor the obvious in this citation. We are now, of course, thoroughly informed of the importance of combined sewer overflows and the need for their control.

There is still one additional question we might ask: why were we so slow in recognizing the potential pollutional importance of urban storm runoff apart from combined sewer overflows? It was not until the late 1950's that investigations in the U.S. established the substantial contribution of pollution by surface wash from urban areas. Of special interest was the observation that pollutants occasionally persisted without appreciable diminution during the course of the storm. In passing we may note that similar studies were undertaken in Moscow and Leningrad in the 1930's.

An increasing number of investigators are now contributing to our understanding of the problem. We believe we may now see the picture in reasonable perspective. However, it is important that we ask what, if anything, have we still failed to recognize? The magnitude of investments that will be made to control pollution by urban storm runoff makes this question urgent. Perhaps during this week of evaluation we may detect areas of useful inquiry that remain to be explored.

I wish to reemphasize my conviction, shared by many if not all of you, that without an urban stormwater quality prediction model our efforts to keep our waters clean can be only partially successful at best, and at worst - ineffective and highly costly. We would all also agree, I'm sure, that models proposed for general use should be tested thoroughly in demanding field situations. Each of us has an opportunity during this week to examine the evidence, to put probing questions to those who have developed and tested the models, and to identify flaws in system design and testing procedure. This short course, therefore, has a dual purpose: (1) to communicate to you important information relative to a new engineering technology, and (2) to derive a feedback to help strengthen this technology. Let us now listen with care, probe and comment constructively, and learn.

INTRODUCTION

By

Richard Field*

This is the first U.S. Environmental Protection Agency (EPA) sponsored course on urban stormwater modeling; and a good way to start would be to briefly introduce you to the EPA Storm and Combined Sewer Technology Program, and how mathematical modeling is an important integral part of that Program.

Storm-generated discharges have come to be recognized as one of the major water pollution problems in urban areas. In an early U.S. PHS report published in 1964, the nationwide significance of pollution caused by storm generated discharges was first identified. This problem was acknowledged ten years ago by the Federal government when Congress authorized funds under the Water Quality Act of 1965, PL89-234, for the research, development and demonstration of methods for stormwater management. The Storm and Combined Sewer Overflow Pollution Control Program, then under the U.S. Public Health Service and now part of the EPA, was designated to carry out this legislative mandate. The effort is now directed by our Section located in Edison, New Jersey.

Up to the present time more than 140 projects totaling approximatley $90 million have been awarded. EPA's share is about $46 million. Our present FY 75 extramural program budget is $1.2 million.

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*Chief, Storm & Combined Sewer Section, Advanced Waste Treatment Research Laboratory, National Environmental Research Center-Cincinnati, U.S. Environmental Protection Agency, Edison, NJ 08817.

EPA STORM WATER MANAGEMENT MODEL

One of the major objectives of the EPA Program is the further development and refinement of mathematical models for the total management of sewer systems which consider the full range of wet- and dry-weather flow pollution control.

In recent years, engineers have found that the traditional analytical methods are inadequate to evaluate the dynamic reaction of the urban drainage system to storm events. It was recognized that a mathematical computer simulation program to model urban runoff quantity and quality during the storm process would provide an invaluable tool for engineers. Such a simulation system was completed in 1971. It is known as the EPA Storm Water Management Model having the acronym SWMM, pronounced "swim".

SWMM is capable of being used in detailed master planning as it enables the decision maker to learn the consequences of alternative courses of action. For example, if storage facilities are being considered, the planner can calculate the hydraulic and the pollutant load that the facility will receive. Similarly, the pollutional load imposed on streams by storms can be calculated. SWMM is also useful in predicting and planning required changes in the hydraulic conveyance capacities of the collection system, brought about by local overloading. For design purposes, the model is useful in determining the anticipated flowrates that a specific sewer needs to convey. The size and slope of the sewer can then be specified. Although the model's operational capability has not yet been applied, there is a likelihood it, or other versions, will be used when storage and treatment works are more widely used in storm and combined sewer systems. The model's operational capability lies in its ability to provide the system operator with information of what to expect at various points in the system in terms of flow and quality from a rainfall event.

The specific purpose of the coursewas to introduce the concepts and modeling techniques associated with the prediction, control, and management of stormwater discharges and combined sewer overflows to participants from government agencies and consulting engineering firms. The participants were specialists in water quality systems planning, design, and management. Specialists who participated in the development of SWMM and other persons who have been active in applying the model or in developing additional options to extend the range of applicability for SWMM or other stormwater models, presented lectures at the course. Participants in the course are expected to improve their planning, design, and management abilities to apply stormwater management models upon completion of the course.

The program is now in the initial phase of demonstrating the ability of computer assisted mathematical models to enhance urban stormwater management by analyzing a major combined sewer system, by selecting and designing control and treatment approaches based on cost effectiveness, and by designing a computerized means of overall management of the system during storm flows. It is the eventual goal to handle all wet- and dry-weather flows in this manner.

EPA FILM "STORMWATER POLLUTION CONTROL: A NEW TECHNOLOGY"

The film[1] portrays a complete overview of the U.S. EPA's involvement in developing countermeasures for pollution from combined sewer overflows and stormwater discharges.

The film starts out with a background of sewer construction which logically leads into the pollution problems caused by urban runoff. Early drainage plans made no provisions for storm flow pollution impacts. Various countermeasures are shown that are ready for implementation by municipalities. The majority are full-scale operations. They include: (a) control techniques, e.g. infiltration prevention, street cleaning, porous pavement (for runoff attenuation, in-sewer routing and storage, improved overflow regulators, and off-line storage; (b) physical, physical-chemical, disinfection and biological treatment shown as adjuncts to the sanitary plant or as remote "satellite" facilities at the outfall; (c) and various schemes which reclaim stormwater for beneficial purposes including enhancement of visual aesthetics, recreation and water supply.

A couple of years ago we felt that the time was right to make an evaluation of previous program efforts, and we implemented a contract to develop the state-of-the-art and assess techniques available to manage and treat combined sewer overflow and stormwater. A text[2] on this subject has been completed and will be available shortly. Expanding on the cliche "a picture is worth a 1,000 words," we felt a film would be worth 100,000--so a film was included as part of the work.

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[1] "Stormwater Pollution Control: A New Technology," U.S.Environental Protection Agency. A 16mm sound-color film available from the National Audivisual Center (GSA), Washington,DC 20409.(Prices: Purchase - $119.50; Rent - $12.50)

[2] "Urban Stormwater Management and Technology: An Assessment," EPA-670/2-74-040, U.S.Environmental Protection Agency. Available from the Storm & Combined Sewer Section, Advanced Waste Treatment Research Laboratory, Edison, NJ 08817.

The specific reasons for this film were to alert the environmental engineering community to the stormwater problem and the immediate need to apply available technology to counteract it.

ENVIRONMENTAL IMPACT OF URBAN STORMWATER RUNOFF

By

Murray B. McPherson*

INTRODUCTION**

All the basic engineering methods for in-stream flood regulation are applicable to flood mitigation works within local urban catchments, but generally on a smaller scale: acceleration of flood flows by canalization through threatened reaches with consequent reduction in stages; isolation of contiguous land in threatened reaches from flood flows by means of embankments, such as levees and flood walls; and the attenuation of flood peaks by means of storage, located either upstream from threatened reaches or at lateral points fed by diverted flows. Similarly, possibilities also exist for social-economic-administrative-legal controls to preclude some degree of damages, such as flood-plain zoning, flood-proofing of structures, flood insurance and related schemes.

Flow in urban drainage conduits is principally by gravity. As for natural drainage basins, smaller sewer branches unite with larger branches, and so on, until a main sewer is reached. The smallest catchment area, on the order of a fraction of a hectare in size, is that tributary to a street inlet. For most smaller tributary areas in the upper reaches of an urban drainage system, the time required to reach peak runoff after the beginning of a storm is a matter of only a very few minutes. Hence, high-intensity, short-duration convectional rainfall is normally the main type of precipitation contributing to the largest runoff rates (in the majority of U.S. metropolitan areas). However, magnitudes of pollution loads are also a function of land-management practices and antecedent precipitation, and maximum loads might not coincide with maximum runoff. Further, because all pollution loads are of concern, cyclonic storms are also relevant.

---

*Director, Urban Water Resources Program, ASCE

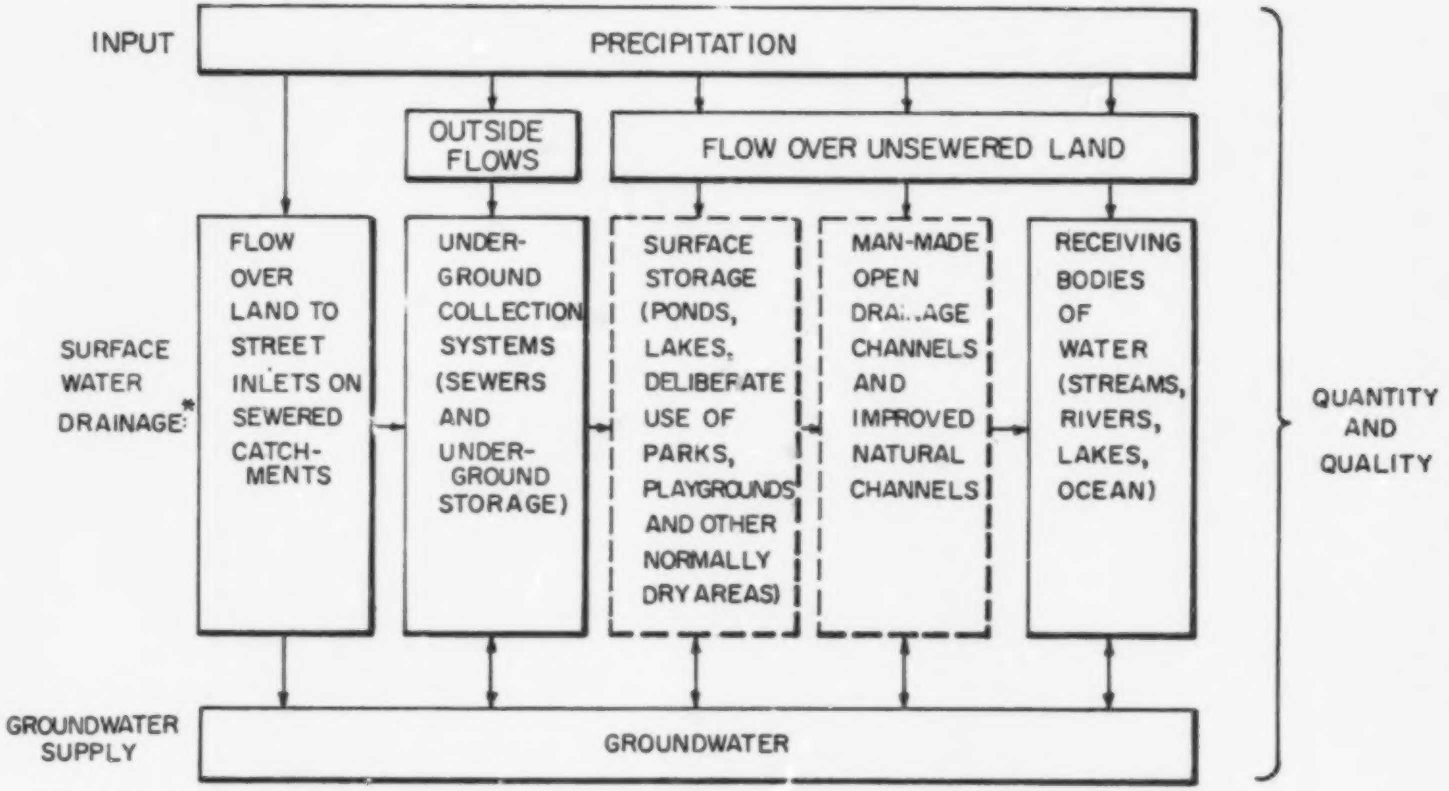
**From "Urban Runoff," by Murray B. McPherson, Chapter 1, Part III, "Illustrative Special Topic Studies," in Hydrological Effects of Urbanization: Environmental Impact, an IHD report to be published by Unesco in August, 1974.

Rainfall interception by vegetation seldom has an important effect on urban runoff magnitude. Most soil infiltration-capacity curves approach a steady, minimum rate after one or two hours, and the capacity of a vegetal cover may be several times as great as for a bare soil. Antecedent precipitation can affect soil infiltration capacity, but very few quantitative data are available for evaluating this factor.

Some of the precipitation which reaches roofs, pavements and pervious surfaces is trapped in the many shallow depressions of varying size and depth present on practically all urban surfaces. There have been no field measurements of depression storage because of the obvious difficulties in obtaining meaningful data.

The residual or excess precipitation remaining after infiltration and depression storage have been abstracted is available to detention, the storage effect due to overland flow in transit. Overland flow refers to surface runoff across a sloping-plane surface at unsteady state, which occurs over extensive portions of an urban area in conjunction with storm occurrences. Overland flow from the land is usually collected in street gutters or ditches, which in turn are drained by street inlets. Additional storage effects occur in the gutters or ditches. Considerable research attention has been accorded the development of inlet hydrographs, but progress has been handicapped by a scarcity of suitable field data.

Flow from the ground surface enters underground systems of conduits at street inlets. The volume of detention in a conduit can effect a reduction in the peak rate of flow of an input hydrograph in the same basic way as any detention storage attenuates an inflow hydrograph, and storage routing hydrographs are used. Advances in conduit routing capability are outpacing available reliable data for monitoring the suitability of techniques developed. This is not to say that the advances have not been significant, but only to emphasize a disadvantage that is encountered universally. Shown schematically in Figure 1(1) is a simplified description of the major


INPUT
PRECIPITATION
OUTSIDE FLOWS
FLOW OVER UNSEWERED LAND
SURFACE WATER DRAINAGE*
FLOW OVER LAND TO STREET INLETS ON SEWERED CATCHMENTS
UNDERGROUND COLLECTION SYSTEMS (SEWERS AND UNDERGROUND STORAGE)
SURFACE STORAGE (PONDS, LAKES, DELIBERATE USE OF PARKS, PLAYGROUNDS AND OTHER NORMALLY DRY AREAS)
MAN-MADE OPEN DRAINAGE CHANNELS AND IMPROVED NATURAL CHANNELS
RECEIVING BODIES OF WATER (STREAMS, RIVERS, LAKES, OCEAN)
QUANTITY AND QUALITY
GROUNDWATER SUPPLY
GROUNDWATER

(* EVAPOTRANSPIRATION NEGLIGIBLE DURING PRECIPITATION)

FIGURE 1-URBAN STORMWATER DISPOSAL PHYSICAL SUBSYSTEM.(1)

components of the urban stormwater disposal physical subsystem. In a given instance, either or both of the two components indicated by dashed lines might be absent.

Whereas there is a continuum between the subsystems of water supply, water use and wastewater reclamation, stormwater has historically been regarded as a purely negative good or nuisance and its subsystem has seldom been deliberately connected to the other urban water subsystems.

Historically, urban settlements have been drained by underground systems of sewers that were intentionally designed to remove stormwater as rapidly as possible from occupied areas. Substantial departures from that tradition are required by new national priorities: enhancement of urban environments; conservation of water resources; and reduction in water pollution.

The greatest public concern will increasingly be on the quality of water. This concern is intimately related to acknowledged imperatives of aesthetic enhancement, expansion of recreational opportunities and more extensive availability of waterfronts for public uses. Runoff is a carrier of wastes, either as harvested for water supplies and converted to waterborne sewage or as an urban ground-surface ablution. Thus, public health considerations can transcend or temper economic considerations. In addition, comprehensive approaches for managing water pollution problems require that other water uses, planning, and guiding sound development, also be considered.(2) For example, utilization of the "blue-green" development concept, which employs ponds with open space, for stormwater detention and recreation, can enhance urban property values and decrease property depreciation rates, thereby increasing long-term local government revenues.(3) On the other hand, peak drainage runoff rates can be reduced by means of proper land-development design.(3) The guiding principle is to reduce the liabilities and increase the assets of urban runoff.(4)

For historical reasons, about one-fifth of the nation's population (or, about three-tenths of the population served by public wastewater collection systems) is served by combined systems of sewerage.(5) Overflows from combined sewers are thought to comprise a significant source of stream pollution.(6) Combined sewers have the dual functions of removing ephermerally occurring storm water from urban surfaces and conveying wastewater on a perennial basis. Conduit size is governed by storm drainage requirements, capacity requirements for wastewater being comparatively small. For all but the relatively few days a year that storm runoff occurs, the continuously flowing wastewater is intercepted near the combined sewer outfall by means of a regulating device which diverts it to the treatment plant via an "interceptor" sewer. Most storm flows are much too great to be accommodated by interceptors, and almost all the wastewater burden is discharge through the outfall to the watercourse when rainstorms are heavy and prolonged. At the same time, sludge and debris that have been stranded in combined sewers during relatively low rates of flow in preceding dry-weather periods are scoured from the laterals and trunk sewers of combined systems and are lifted or otherwise transported by the augmented flows and eventually discharged to the receiving waters. It is estimated that, in consequence, as much as 5 percent of the annual flow of sewage, and 20 to 30 percent of the annual volume of solids, are discharged to receiving watercourses from combined systems.(7)

Pollution from storm sewer discharges(8) may be almost as severe as that from combined sewer overflows. The U. S. Environmental Protection Agency advised in 1971 that requirements for control of pollution from combined sewer overflows were rapidly becoming more stringent and that control of pollution caused by urban stormwater discharges was on the horizon.(9) More recently, the U. S. Council on Environmental Quality has noted that the contribution of pollution from runoff sources is even greater than had been suspected, from both diffused urban and non-urban sources.(10) When abatement of pollution from storm water conveyed by separate systems of storm drains is attempted, the difficulties to be overcome may be more severe because all such storm water must be passed through new and special treatment facilities, there being no interceptor sewers in separate stormwater systems to divert small storm occurrence flows to perennially operated wastewater treatment plants as in combined systems.

In sum, there is widespread interest in multi-purpose drainage facilities that exploit opportunitites for add-on water-based recreation, that provide more effective protection of buildings from flooding, and admit use and reuse of storm water for water supply. These all require storage utilization for effectuation, and some require special treatment plants, necessitating employment of some kind of control system for managing the sudden and brief impact of stormflows. The scale of the problem is almost overwhelming: most of the larger metropolises have well over a hundred catchment areas and cumulative drain lengths of several thousand kilometers in length. The difficulties to be overcome were compounded by the 1972 Amendments to the Federal Water Pollution Control Act which established a national goal of zero-pollution.

LAND-USE CHANGES

Little of the land occupied by metropolitan areas has not been drastically altered by urbanization, particularly the land in large central cities. For example, the distribution of land use in the City of San Francisco is approximately as follows:(11)

Classification	Percentage of Total Area of City
Residential	30%
Streets	25%
Public (about half recreational)	23%
Vacant	8%
Commercial	5%
Industrial	5%
Utility	3%
Institutional	1%

Distribution of land use between a central city and its contiguous metropolitan area can differ appreciably. For example, the proportion of land occupied by residences is commonly higher in the suburbs. Furthermore, there is considerable variation in land use from one metropolis to another. Contributing to this variability are dissimilarities in growth rates. Ranges in land use among seven of the largest U. S. metropolises are as follows:(12)

Land Use	Minimum Portion	Maximum Portion
Residential	1/3	1/2
Roads and Streets	1/6	1/3
Open Space and Recreation	1/20	1/4
Commerical, Industrial, Institutional and Mass Transportation	1/8	3/10

Urban expansion absorbs an estimated 170,000-hectares of land each year in the U.S.(13) Metropolitan suburbs will be particularly hard-pressed to expand all public facilities in the face of a doubling in residential construction across the country anticipated for this decade.(14) Nationally, suburban populations of metropolitan areas had surpassed those of the surrounded central cities by 1970, and this sprawling trend is expected to continue. Of considerable importance to urban runoff management is the usual rapid diminishment in population density with distance from the densest centers. For example, in 1960 the urban population around the City of St. Louis was one-fifth larger than that of the City, but distributed on over four times as much land area.(15)

Buildings, streets and other urban land coverings inhibit the access of precipitation to the soil. The presence of extensive impervious surfaces is thought to be the main reason why the total volume of direct storm runoff from urban areas is generally greater than for comparable non-urban catchments.(16) For example, greatly increased volumes of direct runoff associated with urbanization growth have been documented for some streams in Long Island, New York.(17) Although an indirect indication, evidence has been documented of a correlation between the degree of imperviousness and the post-urban enlargement of stream channels for a number of small watersheds in the Philadelphia metropolitan area.(18)

Estimates of imperviousness ranges for typical urban development are as follows:(19)

Land Use	Percent Imperviousness
Low Density Residential	20-30
Medium Density Residential	25-35
High Density Residential	30-40
Business-Commercial	40-90
Light Industrial	45-65
Heavy Industrial	50-70

Another estimate of imperviousness for typical urban development has also been offered:(20)

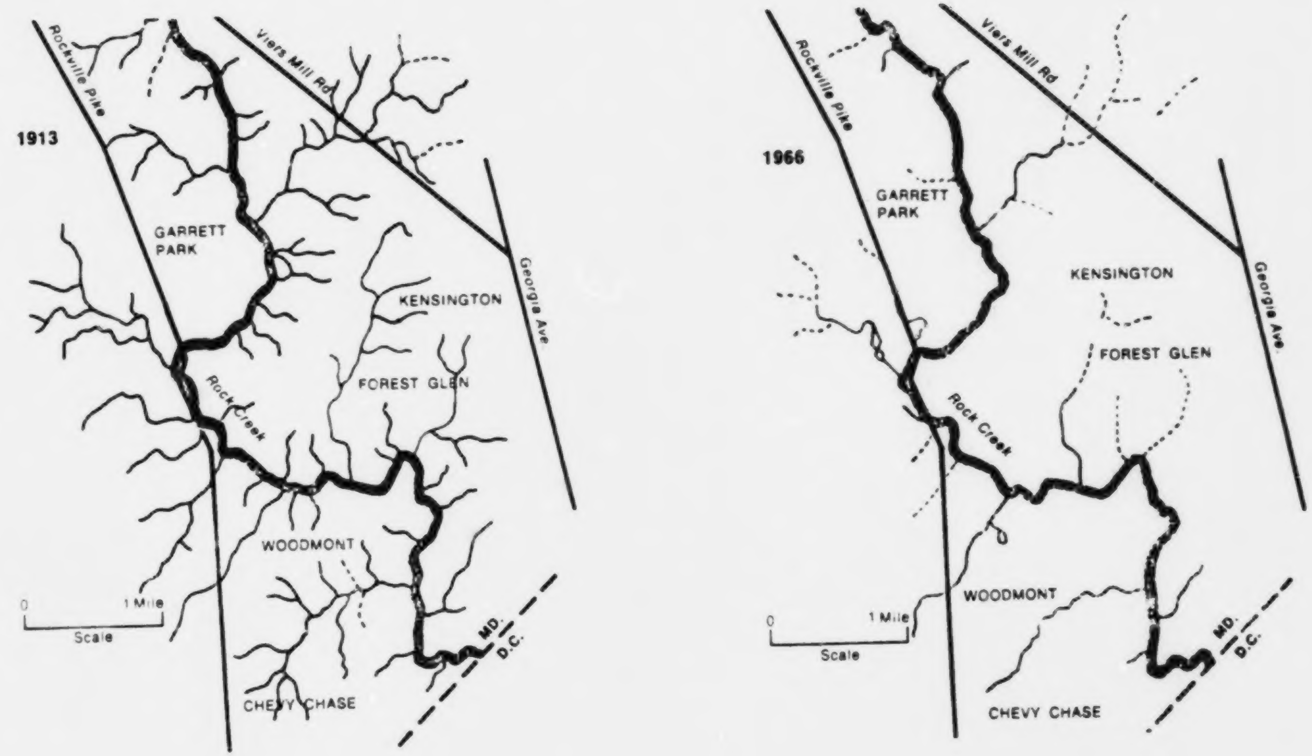
Land Use	Percent Imperviousness		
	Low	Intermediate	High
Single Family Residential	12	25	40
Multiple-Family Residential	60	70	80
Commercial	80	90	100
Industrial	40	70	90
Public and Quasi-Public	50	60	75

Thus, it is not uncommon to find individual downtown city catchments that are almost entirely covered by impervious surfaces and outlying suburban catchments with as little as a tenth as much impervious cover. It follows that the effect of imperviousness on direct storm runoff volume can vary over a considerable range from one urban catchment to another.

For the State of New Jersey, it has been found that urban and suburban land-use characteristics are closely related to population density.(20)

MORPHOLOGICAL CHANGES IN DRAINAGE

Figure 2(21) indicates the diminishment of tributary channels over a 53-year period in a 68-km.^{2} section of Maryland of the Rock Creek watershed, which is now a heavily populated suburban area adjacent to Washington, D.C.



1913
Rockville Pike
Viers Mill Rd
Georgia Ave.
GARRETT PARK
KENSINGTON
FOREST GLEN
Rock Creek
WOODMONT
CHEVY CHASE
MD.
D.C.
0
1 Mile
Scale
1966
Rockville Pike
Viers Mill Rd
Georgia Ave.
GARRETT PARK
KENSINGTON
FOREST GLEN
Rock Creek
WOODMONT
CHEVY CHASE
MD.
D.C.
0
1 Mile
Scale

FIGURE 2- DIMINISHMENT OF TRIBUTARY CHANNELS WITH URBANIZATION(21)

In 1913 this section was rural, with many small tributaries fed by springs and seeps, most of which were covered over as storm sewers were installed. Of the 103-km. of natural flowing stream channels that existed in 1913, only 42% could be found aboveground in this section in 1966.

The function of underground drainage conduits is to remove storm water from urban surfaces (except combined sewers, which in addition convey wastewater on a perennial basis). The smallest catchment area (on the order of a fraction of a hectare in size) is that tributary to a street inlet. Flow in storm and combined sewer systems is principally by gravity. Like natural drainage basins, smaller sewer branches unite with larger branches, and so on, until a main sewer is reached. Thus, a main sewer not only transmits upper reach flow to a receiving watercourse but serves as a collector of surface runoff all along its route. There are over 300,000-km. of storm and combined sewers in the U.S.

From the foregoing, it is therefore not surprising to find that the total lengths of underground drainage conduits dwarf those of open watercourses in major cities. For example, total lengths in the 246-km.^{2} of the City of Milwaukee as of the beginning of 1970 were as follows:(22)

Lakefront Length - 13-km.
River Lengths - 60-km.
Combined Sewers - 885-km.
Storm Sewers - 1320-km.

(In addition, there were 1105-km. of sanitary sewers).

Examples of underground conduit drainage density in some major cities are given in Table 1. Ratios of total sewer length in km. to city area in km.^{2} range only between 8 and 18, except for Los Angeles which makes much more extensive use of open channel drainage than the other cities cited. Also shown in Table 1 are ratios of total sewer length in km. to the 0.6-power of the city area in km.^{2}, ranging between 81 and 136, except for the low value of 20 for Los Angeles. The corresponding

TABLE 1

DENSITY OF UNDERGROUND DRAINAGE CONDUITS IN SOME MAJOR CITIES

City	Area of City, A, Km^2	Total Length of Storm and/or Combined Sewers, L, Km.	L/A Km./Km.^{2}	L/A$^{0.6}$ Km./(Km.^{2})$^{0.6}$
Boston, Mass.[a]	124	2,190	17.6	121
Chicago,Ill.[a]	580	5,786	10.0	127
Detroit,Mich.[a]	360	4,656	13.0	136
Los Angeles,Cal.[b]	1,191	1,389	1.2	20
Milwaukee,Wis.[c]	246	2,205	9.0	81
New York,N.Y.[d]	829	6,650	8.0	118
Philadelphia,Pa[e]	337	4,023	12.0	122
Saint Louis,Mo[a]	160	1,796	11.2	85
San Francisco,Cal.[a]	114	1,400	12.3	82
Washington,D.C.[a]	158	2,816	17.8	135

a: Hallmark, Dasel E., and John G. Hendrickson, Jr., "Study of Approximate Lengths and Sizes of Combined Sewers in Major Metropolitan Centers," ASCE Combined Sewer Separation Project, Technical Memorandum No. 4, ASCE, N.Y., N.Y., May 1, 1967. (NTIS Id. No. PB 185 999).

b: Bauer, W. J., "Economics of Urban Drainage Design," Journal Hydraulics Division, ASCE Proc., Vol. 88, No. HY6, Paper 3321, November, 1962.

c: Prawdzik, T. B., "Environmental and Technical Factors for Open Drainage Channels in Milwaukee," ASCE Urban Water Resources Research Program, Technical Memorandum No. 12, ASCE, N.Y., N.Y., February, 1970 (NTIS Id. No. PB 191 710).

d: (Unpublished notes of ASCE Combined Sewer Separation Project, November 15, 1967).

e: Radziul, J. V., C. F. Guarino and W. L. Greene, "Combined Sewer Considerations by Philadelphia," Journal Sanitary Engineering Division, ASCE Proc., Vol. 96, No. SA1, Paper 7050, February, 1970.

ratio would be only about 2 for river and stream channels,(23) tending to support the notion that underground storm drainage systems replace mostly the smallest natural channels.

The catchment sizes of sewered drainage areas are generally much smaller than those of the natural streams passing through major urban areas, as implied by the examples in Table 2.(24) The small median sizes further substantiate the contention that underground drainage systems replace mostly the smallest natural channels, because natural catchment boundaries mostly tend to be preserved when underground drainage systems are provided.

A survey of combined sewer sizes in major cities revealed that an average of seven-tenths of cumulative city system lengths are 60-cm. (24-in.) or smaller, and that this ratio increases for smaller cities.(25) Because design capacity criteria for combined and storm sewers are very similar, it appears safe to conclude that, for both types collectively, an average of seven-tenths or more of total system lengths are 60-cm. or smaller in size. The typical design capacity(26) of a 60-cm. storm or combined sewer is not much more than about 0.3-cu.m./sec., indicating that the bulk of storm drainage systems convey very modest rates of flow, thus adding further substantiation to the surmise that because the majority of catchments are of small size and therefore collect mostly storm sheetflow, they replace mainly fringe tributary natural channels.

CHANGES IN FLOOD CHARACTERISTICS

The small size of tributary catchment area served by the majority of storm drainage conduits and their small hydraulic design capacity are heavily countered by the large number of such catchments and the fact that they, together with the larger catchments, commonly drain collectively as much as three-fourths or more of metropolitan surface areas. For example, from seven-tenths to nine-tenths of the total areas of the cities of Milwaukee, Washington, San Francisco and Philadelphia are drained by sewered catchments, compared with only about three-tenths of Houston.(24) Because very

TABLE 2

STORMWATER DRAINAGE-CATCHMENT AREA-SIZE DISTRIBUTIONS FOR SOME MAJOR CITIES

City	Total Area of City, Sq.Km.	Number of Catchments	Largest Drainage Area, Hectares	Average Drainage Area, Hectares	Median Drainage Area, Hectares
San Francisco, Cal.	114	42	1750	227	77
Washington,D.C.	158	93	2500	152	26
Milwaukee,Wisc.	246	465	740	39	10
Houston,Tex	1150	1283	1030	26	2

few flow gagings have been conducted at conduit outfalls over hydrologically significant periods, there are few, if any, trustworthy generalizations that can be made specifically about sewered catchment flooding. Historically useful data essentially has been secured from stream gages on watersheds that have undergone varying degrees of urbanization. Examination of available data and of the results of various studies has indicated that the most dramatic hydrologic impact of urban development is that on peak flows, where the basin lag time (or time of concentration) "is reduced as an area becomes urbanized, and the stormflow often is concentrated in sharper, shorter, higher peaks than those for natural runoff."(27)

The very rapid response of a sewered catchment is illustrated in Figure 3.(28) The hydrograph shown was obtained at the outfall sewer exit of a 19-hectare drainage area using a Parshall flume. Response time is so short for such a small but typical catchment that an observation interval no longer than one-minute is required to obtain sufficiently detailed data. Rapid response and serious difficulties in making flow measurements within sewer systems,(29) underground, have presented severe obstacles in the acquisition of basic data.

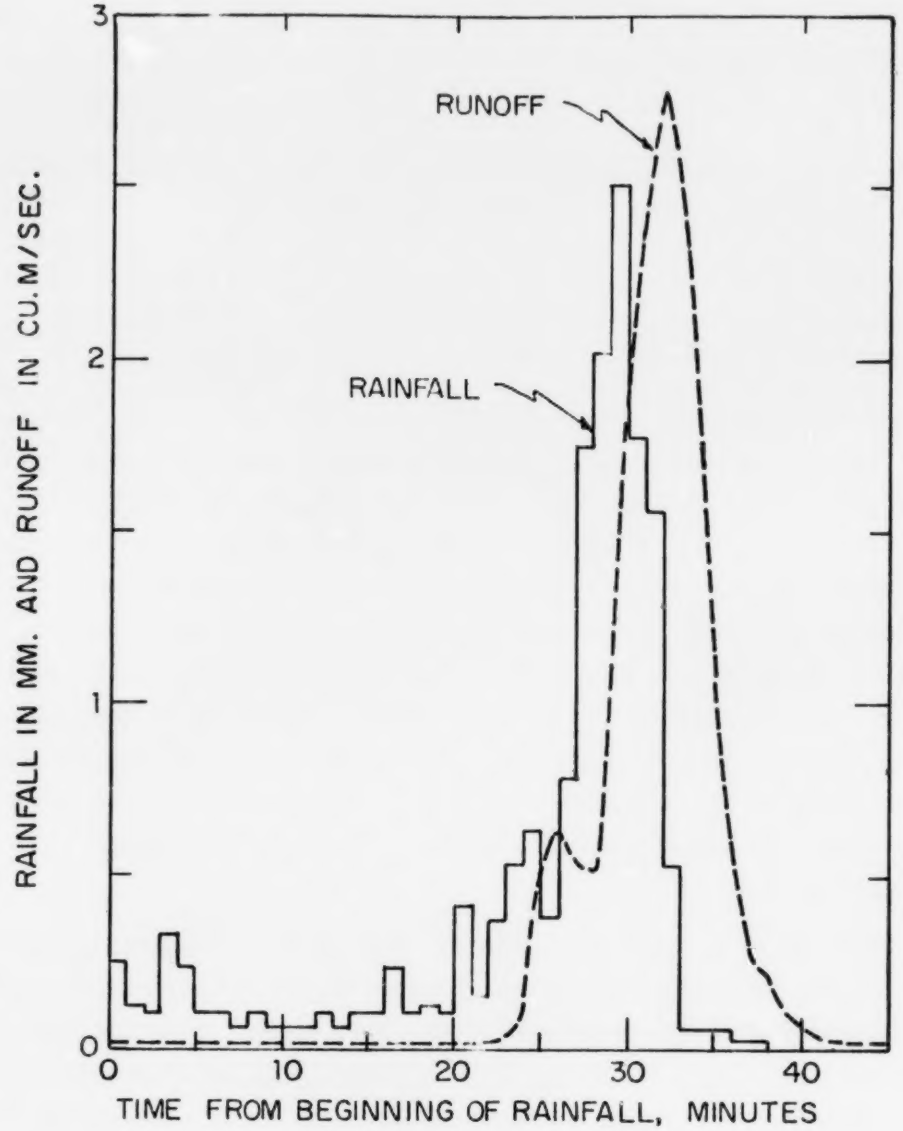


FIGURE 3 - HYETOGRAPH AND HYDROGRAPH, STORM OF 4 AUGUST 1965, 19-HECTARE NORTHWOOD DRAINAGE AREA(28)

Over a period of two decades the portion covered by impervious surfaces grew from 17% to 31% in an urbanized 356-km.^{2} stream watershed located in the Atlanta, Georgia, metropolitan area. Analysis of rainfall and runoff records for the period imply that storm runoff volume has increased in dry months, base flow has decreased in wet months, and peak runoff from summer storms has increased significantly.(30) The latter implication was based on observed changes over time in unit hydrograph characteristics and further substantiates earlier observations in this regard by other researchers.(31)

Man-made underground conduits and the gutters of streets have much lower frictional resistance than the natural minor rills that formed definite watercourses and all the ephemeral channels in the furthermost catchment reaches that they mostly replace. In addition, nearly all major cities often use improved natural channels or excavated special channels for collection of storm water from drainage conduit outlets, such as in Milwaukee,(22) which also expedite the collection and traversing speeds of surface runoff. Because dramatic increases in peak runoff have been documented for some only partially and even lightly sewered catchments, it appears likely that the increased peaks could be caused by the acceleration of flow afforded by the lesser frictional resistance of streets and conduits, although the role of changed aggregate channel lengths is clearly uncertain.

Comparative studies of urban versus rural drainage basins indicate that as the relative magnitude of flood peaks increases the ratio of urban peak rate to rural peak rate declines, the effect of urbanization being more pronounced for the more frequent occurrences.(27) Apparently, grossly overburdened surface and underground collection systems relapse towards the carrying capacity of the natural systems they replace.

Further generalizations are probably not warranted because of the throttling effect of various encroachments, such as loss of natural storage from floodplain development, and the wide variety of drainage and flood mitigation development patterns commonly encountered.(16)

FLOOD MITGATION VERSUS THE AMENITIES OF DRAINAGE

Geohydrologic processes have formed natural drainage channels that convey storm waters to the seas. Because of an early dependence by commerce on water transport, most large metropolitan areas originated as urban centers on or near streams, lakes, estuaries or seacoasts. Intrusion of urban development on natural flood plains has resulted in damages to occupying structures and sometimes loss of life. However, as argued in previous sections, the bulk of urban development in most regions of the nation is located on land that was originally subjected to sheet-flow runoff collected locally in minor channels. Subterranean systems of conduits faciliititate human occupancy by draining sheet-flow runoff from the land surface. That is, fluvial drainage areas contributing to urban flood-plain inundation are often of gigantic size compared with individual underground conduit catchment areas which rarely exceed 2600-hectares in extent. Thus, underground drainage systems are more of a convenience or amenity than a preserver of public safety. Structural means for mitigating flood-plain inundation are designed to provide a much higher level of protection than that for storm drainage systems because of the much greater threat to human life and more community-wide economic implications.

Urban drainage facilities are generally owned, operated and maintained by local governments, and designed and constructed by local governments and private land developers. Human life is seldom threatened by the flooding of these facilities. Because the principal local detrimental effects of flooding are damage to the below-ground sections of buildings and hindrance of traffic, the consequences of flooding range from clearly assessable property destruction to annoying inconvenience. It follows that provision of complete protection from flooding can only rarely be justified. Instead facilities are designed which will be overtaxed infrequently. However, because of the marginal level of protection afforded, storm drainage flooding damages are also of considerable magnitude, probably exceeding those in urban flood plains. In addition, intangible damages from flooding are much more extensive than for stream flooding and generally recur more often, and direct damages are usually much more widely dispersed throughout a community.

The safety of people and properties occupying flood-prone areas is the concern of every level of government, and National, State, special district and local governments are involved in the development of an ever-greater arsenal of remedial or mitigative measures and policies. However, most stream flood-management is undertaken by national agencies. Protection against "upstream" floods experienced on creek and headwater areas is afforded primarily by the Soil Conservation Service, whereas the Corps of Engineers concentrates more on "downstream" floods on mainstems and major tributaries. A notable exception is the Tennessee Valley Authority, which has primary responsibility for both aspects in its region. "A complete inventory of nationwide flood damages has never been undertaken. ....Current efforts to manage flood-plain use and development on a national basis are increasing."(32)

Actual national average stream flooding damages are currently estimated at more than $1000-million annually for urban and non-urban flooding collectively, of which about half occurs in urban areas.(33) In comparison, the estimated actual national average annual loss from the flooding of urban areas served by underground drainage (storm and combined sewer) systems is at least $1000-million.(34) That is, flood damages assignable to sewered catchments may exceed substantially those in urban flood plains. Further, national investment for storm drainage conduit facilities appears to be at least three times as great as that for flood plain protection works benefitting urban areas. The dominance of storm drainage investment over stream protection costs is expected to continue. While the need for more stream protection will rise as greater urban sprawl leads to inevitable new encroachments on flood plains, the same urbanization is expected to require as much as a doubling in the extent of storm drainage systems by the end of this century.

Perhaps only about one-sixth of metropolitan territory is within 100-year flood plains(33) versus well over half of that territory being served by systems of underground drainage conduits. Thus, flood damages per unit of area are clearly greater in flood plains than in sewered sectors of metropolitan areas.

Increased volumes of direct runoff from underground drainage conduits clearly can aggravate flooding of urban flood plains. On the other hand, increased receiving-stream stages can cause or induce flooding of underground drainage systems, because of the intimate hydraulic linkage between them. Extensive programs of integrated flood plain management are particularly crucial for metropolitan areas that have little topographic relief, such as greater Chicago.(35) There are large cities that early dedicated most of their flood plains over to parkways, such as Philadelphia, or developed extensive systems of major drainage channels in step with urban development, such as Los Angeles.(36,37) Regardless, new suburbs have more than occasionally aggravated flooding with subsequent induced damages in many major cities located on large streams or bodies of water because their low topological elevation makes them hydrologically subservient.

There are several non-structural means for mitigating flooding damages resulting from the tenacious inclination of people to occupy the flood plains of urban streams, but these require changes in the activities of occupants and are therefore socio-political in character. The proclivity of home-owners to remain on flood plains has been demonstrated in a pilot study of occupant motivation, hazard perception and attitude, and receptivity towards various forms of communication designed to discourage further encroachment.(38,39)

In sum, discharges from conventional storm drainage facilities and flood-plain intrusion by structures both tend to aggravate flooding, and thereby jointly tend to raise the potential for flooding damages. Revising storm sewering criteria, such as by including much more in-system storage, can be an effective adjunct in flood plain management. While there is universal agreement that planning and development of drainage systems and flood plain management programs should be coordinated and integrated, prospects for accommodation diminish rather than improve because of an increasing concern over water quality considerations in sewered systems and a contemporary neutrality or indifference on water quality matters by agencies dealing predominantly with flood plains. It is ironical that much of the flood plain flooding problem as well as the land runoff water quality problem could possibly be more effectively countered on the land feeding urban watercourses.

Lastly, while the principle of the use of local detention storage is often championed in lieu of main channel improvements for metropolitan areas, sharp differences of opinion have been known to arise, partly or perhaps principally because implementation of the two methods commonly falls within different jurisdictions of authority.(40)

POLLUTION EFFECTS*

Wide recognition of the deteriorating quality of natural waters and large-scale efforts to control waste discharges in the U.S. date essentially from the end of World War II.(41)

"The concept that management of the quality of water is quite as important as its physical management is now widely recognized and advocated in the United States. However, it is of such recent origin that no precise definition of the term has been established and widely accepted, principally because the jurisdiction in which the responsibility for management falls is exceedingly fragmented."(42) The most recent philosophy of public objectives relates to the nature of water as a resource, with an emphasis on its capability for meeting a variety of social objectives.(42) Such orientation moves away from an almost exclusive emphasis on control of quality-depreciating agents as an end in itself. There has been a definite shift from "pollution control" to "quality of the environment" as the objective of national policy.(43) The Federal Water Pollution Control Act Amendments of late 1972 set forth a national goal of zero pollution.

Using broad, collective, subjective indices, estimated U.S. stream pollution causes divide as follows: industrial wastes (trade wastes), one-third; municipal wastes, one-third; agriculture, one-fifth; water management practices, construction, navigation and recreation, three-fiftieths; mining, one-twentieth; and other urban wastes and power generation the remaining close to one fiftieth.(44) The division of pollution load among these sources varies widely with location. The same assessment of the prevalence of pollution found that almost a third of U.S. stream lengths are characteristically polluted.(45) Incidence of polluted streams is highly correlated with major urban population concentrations,(46) as the preceding estimated allocation of causation implies. Attention to subsurface water pollution has not been neglected.(47)

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*From "Hydrological Effects of Urbanization in the United States", by M. B. McPherson, Chapter 6, Part II, "National Case Studies for Selected Countries", in Hydrological Effects of Urbanization: Environmental Impact, an IHD report to be published by Unesco in August, 1974:

Slightly more than two-thirds of the nation's population is served by public systems of wastewater sewerage, with about 46 percent of these people connected to systems with treatment plants that are overloaded or in need of major upgrading and with 7 percent residing in communities with no treatment facilities whatever.(48) However, four-fifths of the total SMSA population is served by wastewater sewerage.(49) While about 55 percent of the volume of wastewater processed by municipal plants comes from homes and commercial establishments and the other 45% is from industries,(50) the latter is only a small part of the total urban industrial burden. "Industries discharge the largest volume and most toxic of pollutants ... Major water-using industries are believed to discharge, on the average, about three times the amount of (settleable and suspended solids and oxygen demanding organic materials) as is discharged by all of the sewered persons in the United States .... The volume of industrial wastes is growing several times as fast as that of sanitary sewage ...."(48) The fact that potential water supply sources are evaluated with regard to levels of quality as well as adequacy of quantity means that the choice of source ultimately can be dictated by quality considerations, and thus quality rather than quantity alone can determine the location of source impact on the hydrologic balance of a subregion.

The surface and underground waters of the nation, the present focus of water pollution abatement activities, are the "raw" sources for drinking-water supplies. The most demanding quality requirements are for the transformation of raw water into drinking water in purification plants and the subsequent delivery of safe water to users. Stream quality objectives have tended to emphasize dissolved oxygen levels and micro-organism contents. Water purification viability is little affected by raw water dissolved oxygen levels, and micro-organisms can be rendered harmless as is evidenced by the essentially unblemished public health record of water treatment and distribution over the past several decades. Rising amounts in raw waters of toxic chemicals, including heavy metals and exotic organics, uncertainties on the instantaneous and cumulative human health hazards of these substances, and resultant ambiguities on treatment removal objectives and standards, are sources of considerable management concern. That water purification must increasingly be more meticulous is widely accepted.(51)

About one-fifth of the nation's population (or three-tenths of those served by community wastewater sewerage) is served by combined systems of sewerage(51) and of the fourteen largest cities, ten have combined systems in whole or in part.(52) The first definitive identification of the pollution problem from combined sewer overflows on a national scale appeared in 1964.(56) It has been estimated that, on the average, where dry-weather flow receives secondary treatment, during storm events overflows from combined sewers represent a load equivalent to about one-third of the treatment plant effluent load, which is therefore about one-fourth of the total load discharged to receiving waters.(53) The average annual BOD load from combined sewer overflows is estimated at roughly 550-kilograms per hectare served, not including additional loads due to treatment plant upset or flows bypassed at plants as a result of storm events.(54)

"Requirements for control of pollution from combined sewer overflows are rapidly becoming more stringent. Control of pollution caused by urban storm water discharges is on the horizon."(59) Documentation of the potential storm drainage (rainwater sewers) pollution problem started in 1964(55) A disproportionate amount of street surface pollution load is associated with fine dust and dirt particles, and a significant fact is that heavy metals and toxic material concentrations are much higher in urban runoff than in domestic sewage.(54) Preliminary findings indicate that polychlorobiphenol concentrations of stormwater range between 0.4 and 2 parts per million; BOD ranges between 1/2- and 17-kilograms per curb kilometer; COD ranges between 4- and 110-kilograms per curb kilometer; and mercury and lead have been found to range between 0.0055- and 0.085-, and between 0.034- and 0.522-kilograms per curb kilometer, respectively.(54) (Tetraethyl lead presently is an additive in most U.S. petrol.)

Because very few sewered catchments have been gaged, and nearly all have been gaged only at outfalls, very little is presently known about rainfall-runoff-quality processes. Much more investigation must be undertaken.(54)

Salting of streets and highways, used for the facilitation of traffic in ice and snow, results in local soil concentrations of leachates.(56,57)

In 1966, pesticide use in the conterminuous U.S. totaled almost one-third of a million metric tons of active ingredients, half of which was used by farmers and the remainder by government, industry and homeowners.(56) Pesticides and herbicides have diffused and concentrated throughout the land-air-water phases and pose direct and indirect threats to the health of urban dwellers and the survival of numerous animal species.(58) One of the first documented studies of the carriage in runoff of pesticides from agricultural catchments was reported in 1966.(59)

Disposal of the residue from wastewater treatment(60) is fast becoming a major problem,(61) with sludge quantities increasing, while use of nearby potential land disposal sites is progressively foreclosed by development, and sea disposal is challenged as a suspected cause of environmental degradation.

Total municipal wastewater solids, most of which are removed in conventional treatment, average on the order of 90-metric tons per day per million persons served, dry weight. However, sludge is often transported to disposal sites as a slurry. Costs of transportation and handling can be reduced by slurry volume-reduction through concentration or "thickening" of sludge solids. Based upon the maximum concentration that can be hydraulically transported, the minimum slurry load is about 360-metric tons per day per million persons served. However, normal practice, which reflects reliability of operation as well as cost-attractiveness, results in a slurry of about 900-metric tons per day per million persons served. The magnitude of the sludge disposal problem can be appreciated by noting that the preceding tonnage is in the neighborhood of half the weight of solid wastes per million persons accumulated per day. As opposed to solid waste disposal, wastewater sludge disposal encounters two monumental liabilities: an extremely large water content and a very high organic composition. As a result, the feasible avenues of disposal generally differ from, and often are more restricted than, those for solid wastes.

Since mid-1971, a special district serving most of the Chicago metropolitan area has been barging by river about half of its daily sludge production in slurry form to an area about 320-kilometers from Chicago.(62) Expected increases in sludge load from the Chicago area will be disposed of in strip mine and farmland reclamation.

Several major coastal cities dispose of sewage sludge by barging it out to sea. Almost 4-million cubic meters of sludge slurry from the New York metropolitan area are disposed of annually in the New York Bight of the Atlantic Ocean.(63,64)

As opposed to the more or less steady regularity of solids content for a given wastewater system, solids in surface water drawn for domestic use often exhibit large seasonal variations (largely from episodic suspended sediments) and can range from such small amounts that no treatment reduction is warranted to concentrations well above those of normal wastewater. Hence, the seriousness of the water treatment sludge disposal problem varies among different sections and localities.(65) On the average, the amount of sludge produced annually from all U.S. water treatment plants is on the order of about 25-metric tons of slurry per million persons served.(66)

Land disposal of wastewater treatment plant effluents as an alternative to conventional water disposal is receiving greater attention as concern for control of surface water quality intensifies.(67,68)

Wherever solid wastes are deposited on land, there will be opportunities for these wastes to intermingle with surface water and groundwater. The extent of pollution of groundwater by the leaching of surface water through deposits of solid wastes is largely dependent upon the character of the immediate geologic environment,(69) but groundwater can be protected by modifying landfill sites.(70,71,72)

Estimated sources of total solid wastes produced in the U.S. in 1969 divide as follows:(73)

Source of Wastes	Million Metric Tons
Residential, commercial and institutional -	230
Industrial -	100
Mineral -	1,500
Agricultural -	2,100

Concentrations of mineral and agricultural wastes are spread widely over the land and are mostly isolated from population concentrations, although some deleterious effects on urban residents are nevertheless suspected, and have been documented in isolated instances. The bulk of the other wastes are disposed of in or near urban areas and represent an ever-present threat to public health. Space available for municipal solid waste land-disposal is fast disappearing. Integrated management of urban residuals in air and water and on the land is regarded as a necessity for adequate environmental protection in the future.(74)

Urbanization and associated human activities have accelerated geological processes of land erosion.(75) Damage is inflicted at the scene where soil is eroded, where it is washed downstream, and where sediment remains suspended or comes to rest.

"Efforts to control sediment pollution must be concerned not only with treatment of water which is already affected by sediment, but also with controlling the massive amounts of soil being washed into waterways each year."(76) Three broad categories of land disturbance are appropriate foci in control efforts: erosion of "natural" or geological character; erosion resulting from agricultural, forestry and mining activities; and erosion associated with suburban development.(76) "Evidence being made available by current research suggests that sediment yields in areas undergoing suburban development can be as much as 5 to 500 times greater than in rural areas."(76) An estimated 170,000-hectares of land per year are taken over by urban expansion.(77)

The sequential qualitative effects of various land-uses on relative sediment yield and channel stability have been described as follows:(78)

Land Use	Sediment Yield	Channel Stability
Natural Forest or Grassland	Low	Relatively stable with some bank erosion.
Heavily Grazed Areas	Low to moderate	Somewhat less stable than the preceding.
Cropping	Moderate to heavy	Some aggradation and increased bank erosion.
Retirement of Land from Cropping	Low to moderate	Increasing stability.
Urban Construction	Very heavy	Rapid aggradation and some bank erosion.
Stabilization	Moderate	Degradation and severe bank erosion.
Stable Urban	Low to moderate	Relatively stable.

Stabilized urban development sediment yields tend to approach presettlement levels,(79) as noted above.

Sediment from erosion impairs water quality, reduces the volume of available water supplies, and damages recreational waters, quantitatively and qualitatively, such as by accelerating eutrophication. Effectiveness of mosquito-control works can be seriously reduced by erosion-deposition occurrences. The biological viability of aquatic biota is inhibited by suspended sediments, resulting in reduced waste-assimilation capacity of water-bodies and other deleterious effects.(80) The "importance of sediments as pollutants is increasing, particularly in view of the ability of soil to absorb pesticides and other organics including oily substances and to release these materials in the water resource. Indeed, this facility of sediments to receive chemicals from the solution phase, when properly understood, may be subject to manipulation and made to serve effectively to trap and immobilize harmful wastes moving in our environment."(81)

In terms of public safety: main-channel deposits can aggravate main-stream flooding; sediment deposited on and in property on the occasion of overbank flooding can cause considerable damage over and above that of inundation; structural integrity of water conveyance facilities, such as culverts and bridge piers, can be imparied or destroyed; and buildings can be badly damaged or destroyed and channel flooding can be aggravated by landslides induced by heavy rains.(82,83,84)

Removal of sediment deposits from public thoroughfares and other public areas adds to the cost of maintenance. Turbidity from mineral and other sediments can tax water treatment facilities.

In sum, few beneficial effects, if any, can be claimed presently for erosion in the urban environment, and the cost of uncontrolled sediment movement is indeed large.

The significant amount of water withdrawn in metropolitan areas for power-generation cooling has been noted in Section 6. If the present growth rate of total energy consumption in the U.S. persists, in about fourteen years it will be double the present amount.(85) Generation of electricity accounts for less than one-fourth of all U.S. energy consumption.(85) Waste heat from electric power generation is one of the most serious emerging sources of water pollution, accounting currently for four-fifths of all the thermal heat entering the nation's waters, and heat discharges increase as generating capacity doubles every decade.(86) The total quantity of waste heat discharged to condenser cooling waters will more than double from the year 1967 to the year 1980.(87) The trend towards larger, nuclear plants, which discharge three-halves as much thermal load in water per unit of power as fossil-fuel plants, pose an increased threat to aquatic systems.(86) Among several alternatives to the use of water bodies as heat sinks is the use of cooling towers, already commonly deployed in Europe but little used in the U.S., much to the ire of conservationists, particularly for nuclear sites.(88) Dissipation of residual heat is regarded as a serious problem for future generating plants.(89,90,91) Considerable attention is being given to the environmental problems of plant siting,(92) but the root source of these and related problems is considered to be the absence of a national energy policy.(93)

National energy and water policies will be shaped by overall growth policy. The National Government in 1971 was attempting to resolve the final determination of a national growth policy,(94) and in 1972 it was asserted that "the time has come for the United States to adopt a deliberate population policy."(95) Because most land-use decisions are made by private interests and governmental control over land-use is fragmented at the local level, better land-use planning at all levels of government is advocated as a part of overall national grow policy.(96) Also needed are regional bodies with authority to plan and control those facets of land use that transcend local boundaries, such as pollution abatement.(97)

BETTER DESIGN OF STORMWATER DRAINAGE SYSTEMS

DESIGN OBJECTIVES

Much has happened since APWA presented its report on urban drainage in 1966(34) and on combined sewers in 1967,(98) and even since the report on storm runoff quality in 1969(99). Historically, urban settlements have been drained by underground systems of sewers that were intentionally designed to remove storm water as rapidly as possible from occupied areas. Substantial departures from that tradition are required by new national priorities: enhancement of urban environment; conservation of water resources; and reduction in water pollution.

The greatest public concern will increasingly be on the quality of water. This concern is intimately related to acknowledged imperatives of aesthetic enhancement, expansion of recreational opportunities and more extensive availability of waterfronts for public uses. Runoff is a carrier of wastes, either as harvested for water supplies and converted to water-borne sewage or as an urban ground-surface wash. Thus, public health considerations can transcend or temper economic considerations. In addition, comprehensive approaches for managing water pollution problems require that other water uses, planning, and guiding sound development, also be considered.(2) For example, utilization of the "blue-green" development concept, which employs ponds with open space, for stormwater detention and recreation, can enhance urban property values and decrease property depreciation rates, thereby increasing long-term local government revenues.(3) On the other hand, peak drainage runoff rates can be reduced by means of proper land-development design.(3) The guiding principle is to reduce the liabilities and increase the assets of urban runoff.(4)

Capturing and/or diverting storm water for recreational and aesthetic enhancement, for use as a water supply supplement, and for meeting the zero-pollution goal of Public Law 92-500 (October 18, 1972), all require or imply the employment of some degree of detention and/or retention storage. Such storage would have to be added to existing systems and included in new systems, at substantial cost.

The Environmental Protection Agency advised in 1971 that requirements for control of pollution from combined sewer overflows were rapidly becoming more stringent and that control of pollution caused by urban stormwater discharges was on the horizon.(9) In 1972 the Council on Environmental Quality noted that the contribution of pollution from runoff sources is even greater than had been suspected.(10)

Discharges from conventional storm drainage facilities and floodplain instrusion by structures both tend to aggravate flooding, and thereby jointly tend to raise the potential for flooding damages. Revising storm sewering criteria, such as by including much more in-system storage, can be an effective adjunct in flood plain management. While there is universal agreement that the planning and development of drainage systems and flood plain management programs should be coordinated and integrated, prospects for accommodation diminish rather than improve because of an increasing concern over water quality considerations in sewered systems and a contemporary neutrality or indifference on water quality matters by agencies dealing predominantly with flood plains.(100) It is ironical that much of the flood plain flooding problem as well as the land runoff water quality problem could possibly be more effectively countered on the land feeding urban watercourses.

Techniques for hydrologic analysis of rivers and urban streams are at a stage of development far beyond those for local drainage, mostly because of a much broader data base.

The procedure used in nearly all current storm sewer design is the "rational method",(26) the numerous inadequacies of which have been discussed elsewhere(101) It is suffucient to emphasize that the method yields only an estimated peak flow but none of the other attributes of a hydrograph. A full hydrograph is needed for the design of detention storage, for evaluation of pollutant burdens, for design of stormwater pollution abatement facilities, for designing local protection works along streams, such as pumping stations for passing local drainage flows over levees and dikes, and as inputs for design of stream and river development works. Also, quantification of the effects of urbanization on the hydrologic regimen is dependent in many cases on the availability of sewer outlet hydrographs. Further, as urban water management problems become increasingly acute, the need for multiple-use of water becomes more evident. In exchanging one use for another, for example by using storm water as a source of water supply, knowledge of the time-histories of flows and water qualities is essential for reliable design of transfer facilities. In sum, there is essentially world-wide agreement that improved methods for design based on field observations are sorely needed.

Analytical requirements for planning are less rigorous and permit less detail than for design because investigation of a range of broad alternatives is at issue. What are sought for planning tools are general parameters or indicators for large-scale evaluation of various alternative schemes. Hence, the degree of computational detail required in metropolitan planning is much less than for design. However, a certain amount of intensive detailed analysis is needed to establish parameters and indicators and to provide an underlying understanding of the governing hydrologic processes, so that simplified expedients are not inadvertently misused.

The ultimate solution for the problem of abating pollution from urban storm sewer discharges and combined sewer overflows is the treatment of such flows prior to their release into receiving waters. Outflow surface-water hydrographs of sewered catchments exhibit very large peaks, on the order of two or more times those of equivalent non-sewered areas.(102) Capturing, transporting and treating all discharges/overflows, unattenuated, would require gigantic collection sewers, pumping stations and treatment facilities - all of which would be used less than the equivalent of about an hour a day, on the average, over a typical year. Further, there will be periods running into several weeks or even months where the discharge/overflow control works will not operate at all, as a consequence of zero or light precipitation.

Therefore, schemes for system-wide discharge/overflow collection and treatment incorporate some form of auxiliary storage for the purpose of attenuating collection and treatment facilities inrushes, with the objective of scaling down the size of such facilities to reasonable and manageable proportions. In addition, the complexity added by converting a facility with simple gravity flow into a multiple-component interacting scheme, where practically instantaneous response to flow incidences must be made, requires incorporation of some degree of automatic operational control. Combined sewer system applications are exemplified by plans for the Chicago metropolitan area(103,104) and for the City of San Francisco.(11,105)

Compared with design and planning, central service objectives stand out in bolder relief in operations: minimized flooding of properties, minimized quality degradation of receiving water bodies, maximized recreational and aesthetic opportunity or maximized water supply use. Whereas process mastery is vital for flexibility in design and planning of project alternatives, the control mode can be, and probably must be, approached in a much more pragmatic fashion.(106) The real test of performance effectiveness occurs in basements of buildings and in contaminant levels of receiving water bodies.

SEWER DESIGN

The "rational method" applies to a very unique and restricted set of assumed conditions. Once a system is designed using the fixed features inherent in the method there is no logical way to analyze modifications, such as provisions for relief, reusing the method. Also, because there is no direct way to verify the method in the field, even the adequacy of the original design cannot be checked. The simplistic scope of the method permits and requires a wide latitude of subjective jdugment in its application. Because the method is based on hypothetical conjecture unsupported by runoff data, generally speaking there can be no "wrong" storm sewer designs when the rational method is employed. An outstanding limitation of the method is that the product is restricted to a peak flow; and although by heaping more assumptions on the method a hypothetical hydrograph can be computed, such as by arbitarily proportioning the contributing area with time, few desgners are willing to elect this frivolous option.

If the method is so inadequate, why has it seemed to "work" all these years? Three factors affect the magnitude of a design flow in using the rational method: C-value, inlet time and the frequency for the rainfall intensity-duration curve used. Computed peak flows are larger as the C-value is raised, as the inlet time is shortened and as a curve for a rarer rainfall frequency is used. Intelligent, though arbitrary, selection of values for these three variables has been found to give ostensibly "satisfactory" results in a number of places. There are several factors which may contribute towards conservatively "safe" designs, such as the usual practice of designing sewers to accommodate at least the design flow rate at a flowing-full condition, whereas some degree of surcharge might be sustained without flooding. Also, the probability of all design assumptions being satisfied simultaneously is less than the probability of occurrence of the rainfall rate used in the design, contributing in effect to a "safe" design with a built-in factor of safety.

Probably the most important influence is the narrow range of input values: the range of short duration rainfall intensities provided by nature. For example, a 20% variation in design rainfall intensities about 5-year values covers a range of about a 2-year to about a 15-year frequency.(101) Stated another way, shifting the C-value up or down by 20% has the effect of changing a 5-year design frequency to about a 15-year or 2-year frequency, respectively.

If all that is wanted is a design peak flow, and if a public works department is not going to make any field rainfall-runoff or rainfall-runoff-quality measurements, the rational method is as good an arbitrary procedure to use as any, considering the primitive state-of-the-art of urban hydrology.

If a design hydrograph is wanted but a public works department is not going to make any field measurements it is recommended that the unit hydrograph procedure synthesized from Louisville data by Eagleson(107) and expanded to include other data by Espey(101) be used, such as in its adaptation for metropolitan Denver.(109,110) The reliability of the computed peak flow would likely be rather poor and not much better than that obtained by the rational method. However, if *alternative* designs are at issue the unit hydrograph approach is more likely to yield a more reliable *difference* between the designs because it takes more land-use characteristics into account. Many departments are faced with questions which require immediate answers, on the effect of sub-catchment land-use changes on total catchment drainage system adequacy, such as the extent of new relief sewer construction that would be required to convert an existing residential sector into a shopping center or other drastic change. There is no reason why the unit hydrograph method should not be used for such applications. Indeed, there is no logical exuse for *not* using that method to examine the merits of one alternative against another in any application.

If a design *hydrograph* is wanted and *if* a department *is* going to make some field measurements, use of a range of tools becomes feasible. These tools have been calibrated using most of the limited amount of data that is available. Outstanding among these, in order of simplicity of interpretation and application, is the unit hydrograph, already cited, methods tested at Purdue(111) and the Road Research Laboratory method.(112) However, the transferability or applicability of these methods to catchments other than those on which these methods were calibrated would be completely speculative unless some local measurements were used to check their approximate local relevance and to sharpen the parameter values involved for the purpose of obtaining more consistent and reliable results.

It is important to note at this juncture that we have gone from the simplistic rational method, which can be intelligently applied by sub-professionals, to methods which require a good understanding of basic hydrological principles and some mathematical transformations plus an ability to conduct field research and interpret the results of the findings for use in design. Even the best of tools in unskilled hands can be badly abused. If a department has a serious commitment to improved or more flexible design it should have at least one well-qualified hydrologist on its staff. Qualifications can be achieved by a good engineer with an initially sparse competence in about a month of full-time intensive self-study. As the discussion in this paper continues, techniques will be mentioned that require still greater capabilities that must be mastered by staff personnel.

Because complex processes, such as in the hydrological response of a sewered catchment to a precipitation occurrence, can never be fully replicated in a computation due to incomplete technical understanding of the processes and the infeasibility of detailing the myriad pieces involved, resort is made to simulation of response of a conceptually equivalent system. The simulation package is commonly called a "model", and this term is even applicable in principle to the primitive "rational method". As you know, models can become quite sophisticated and involved. There are instances of departments that assumed that the more complicated the procedure the better the results. That is, applying a larger number of guesses to a larger number of unknowns is somehow better than a smaller number. Too often the claims of model builders about the universality of their models has been accepted without question. Model builders are to be listened to but not fully believed for they have a vested interest in the survival of their progeny.

Use of models is a function of what you want to use them for, how you want to use them, how much you are willing to invest, how often they would be used, what levels of precision you want, what kind of outputs you want, how much time you can spend in getting the model to work, how much you can commit to calibrate and verify a model, etc., etc. While outside advice can be obtained on the answers to these many questions, in the long run only Department personnel can decide these matters if Department needs are to be satisfied as the Department sees them.

There are several elaborate models that are undergoing continuous upgrading and improvement. Among these are the Stanford Watershed Model(113,114) and the EPA model.(115-118) In the same complexity category are the models developed by Dorsch of the Federal Republic of Germany and by SOGREAH of France, both of which are being promoted in the U.S. Early in 1973, EPA formed a "users group" to create a forum for the exchange of ideas and information relating to the EPA model and its use, and to enable users to benefit from the experience of others in applying or modifying the model to fit particular situations.(119)

Occasionally the assertion is made that the totality of a jurisdiction should be subjected to detailed modeling using a model of the type mentioned immediately above. Most of our major cities have dozens and even hundreds of stormwater drainage catchments with cumulative conduit lengths often exceeding 1000-miles. For example, Milwaukee has 1,370-miles of separate storm drains and combined sewers within the 97-sq. mi. of the City.(22) These conduits are distributed over 465 drainage catchments in Milwaukee having a maximum size of 1,820-acres and a median size of 25-acres.(24) Additional examples are given in Tables 3 and 4. These examples should suffice to indicate that an extremely compelling argument would be required to justify an extremely detailed modeling for the existing facilities of jurisdictions of these large sizes. However, under restrictions already cited, detailed modeling is justified and completely appropriate in the hydrologic-hydraulic design of extensions to existing systems and for altogether new systems.

If a department is make field measurements and is using comparatively short-cut methods (such as the unit hydrograph, Purdue or RRL methods), overall reliability and the physiological meaning of these methods in terms of gaged observations can best be monitored and explained by the occasional use of the more elaborate methods such as the Stanford Watershed or EPA models. Department staff must be faimilar with functional model details, however, or subtleties and aberations will be easily overlooked.

TABLE 3

EXTENT OF UNDERGROUND DRAINAGE CONDUITS IN SOME MAJOR CITIES*(100)

	Area of City, Square Miles	Total Length of Storm And/or Combined Sewers, Miles
Boston	48	1,360
Chicago	224	3,600
Detroit	139	2,900
Los Angeles	460	860
Milwaukee	95	1,370
New York	320	4,130
Philadelphia	130	2,500
St. Louis	62	1,120
San Francisco	44	870
Washington,D.C.	61	1,750

(*Note: Data are not available for total metropolitan areas, but it may be assumed that in most of the above instances the length of sewers in the areas surrounding these cities are much greater).

TABLE 4

STORMWATER DRAINAGE-CATCHMENT AREA-SIZE DISTRIBUTIONS FOR SOME MAJOR CITIES(24)

City	Total Area of City, Sq. Mi.	Number of Catchments	Largest Drainage Area, Acres	Median Drainage Area Acres
San Francisco	44	42	4330	190
Washington	61	93	6180	65
Milwaukee	95	465	1820	25
Houston	444	1283	2550	6

In sum, elaborate models are of limited practical use in massive applications, such as in the preliminary design phase of a master plan, but are invaluable in monitoring and explaining simpler methods and may be useful in detailed final design if department staff are skilled in their use and if local data is available for their calibration. Continual improvements are being made in the more elaborate models, all of which enhance their application potential.

Parenthetically, hybridization of models also may be useful. For example, the water computer sub-routine of the EPA model can be "piggy-backed" on a computer program for the simpler rainfall-runoff models such as the unit hydrograph, Purdue or RRL methods. (Researchers at Purdue added water quality samplers at their rainfall-runoff field measurement stations in 1972, and future data analyses will incorporate quality aspects, perhaps including previously investigated models.) Hydrologic modeling of systems of underground drainage which does not or cannot accommodate water quality considerations must be judged of limited utility under contemporary imperatives. On the other hand, for comprehensive capture and treatment schemes, estimates of pollution loads may be needed only for design in the few occasions when discharges/overflows would occur, and when all occurrences are wanted - as in metropolitan-scale problem assessments - the simplest techniques should be employed because of the enormous scale of the assessment problem. To reiterate, limited and periodic use should be made of more elaborate models to monitor the simpler methods, provided some local data is available for calibration.

CHALLENGES IN THE USE OF MODELS FOR DESIGN

A survey has been made of existing raingage networks.(120,121) In 1969,(120) fifteen of the largest metropolitan areas had a network of recording raingages of between about 5 and 192 instruments, with records spanning periods of 2- to about 50-years. Since 1970, an urban-weather research program(122) with instrumentation that includes 250 recording raingages, the world's first major field-laboratory program aimed at assessing the effect of a major urban-industrial complex on precipitation,(123) has been in progress in the St.Louis metropolitan area. The most advanced automatic network was installed in San Francisco in 1971, where rainfall rates from 30 gages are transmitted at a 15-second interval to a control data acquisition and recording station.(105)

Despite the not inconsequential amount of raingage network data accumulated thus far, advances in the characterization of time-spatial characteristics of storms over urban catchments have been slow in coming.

Human life is seldom threatened by the flooding of urban drainage facilities. Because the principal local detrimental effects of flooding are damage to the belowground sections of buildings and hindrance of traffic, the consequences of flooding range from clearly assessable property destruction to annoying inconvenience. It follows that provision of complete protection from flooding can only rarely be justified. Instead, facilities are designed which will be overtaxed infrequently. A monumental question in the analysis/design of drainage systems is the choice of storms to be used. Storm definitions used for deriving river basin extremes such as "reservoir design floods" and "spillway design floods" are irrelevant because urban sewer systems are expected to be overtaxed much more frequently than major river structures whose failures could be catastrophic. From this standpoint, the mean frequencies of occurrence of flow peaks and volumes and quality constituent amounts is the issue, not the frequencies of the input rainfall, and were it possible to arrive at statistical series for discharge-quality independently of rainfall we could vastly simplify the storm characterization issue.

Furthermore, because there are inherent non-linearities in most methods for processing inputs for linear models, the dynamic models are non-linear by definition, the statistics of the rainfall input array may differ appreciably from the statistics of some or all of the arrays for runoff-quality characteristics. That is, attempting to assign a mean frequency of probable occurrence to a "design storm" is meaningless because of statistical non-homogeneity of rainfall, runoff and quality. Also, such an approach neglects the effect of prior storms on the runoff from a given storm.(124)

In a given application, it is more reasonable to route rainfall data of local record through a model to arrive at rate-volume-constituent frequencies than to synthesize a storm of specious probability. For example, in developing the Master Plan for combined sewer overflow pollution abatement in San Francisco, preliminary indications of overflow frequency were obtained by using hourly rainfalls of the 62-years of record from the City's U.S. Weather Service gage.(11)

Over and above the storm definition problem is the inherent difficulty with any runoff model in the necessarily subjective separation of abstractions (infiltration, depression storage, etc.) from total rainfall to resolve rainfall excess (amount and pattern), which is the input from which an equal volume of direct runoff is generated by models of one kind or another. This problem is greatly aggravated by the extreme scarcity of field data for calibrating and verifying models of all types.

In the operating mode, any control system must not only respond almost instantaneously to the actual occurrence of rainfall but must anticipate the probable character of subsequent time and spatial changes barely before they occur. A desirable adjunct would be an incipient storm-occurrence forecasting capacility. As in design/analysis, separation of abstractions from total rainfall to derive rainfall excess is necessarily highly subjective, and delimits the relability of affected control-response components. While detailed rainfall-runoff-quality modeling based on real field data is both desirable and logical for development of control criteria, existing models suffer substantially from a severe shortage of data for their calibration, verification and, particularly, for their realistic application to non-gaged catchments. As noted earlier, the control mode can be, and probably must be, approached more pragmatically than design/analysis, and the ratio of art to science required for the automatic operational control of urban water services will probably be highest for storm sewer systems.(106) To illustrate the expected pragmatic emphasis, simplified flow-routing methods(125) based on the storage continuity equation will probably find much greater use than complet methods based on simultaneous solution of the partial differential equations of flow. That is, control logic will tend to incorporate simplified flow-routing methods while the validity, sensitivity and reliability of such short-cuts will be monitored by selective use of more complete methods.

MASTER PLAN IMPLICATIONS

Exploration of metropolitan-wide pollution-control alternatives should employ the simplest of mass-balance and mass-routing techniques, because of the scale of the problem and the sparsity of suitable data for more refined analysis. Total metropolitan water-balance inventories(126-128) can serve as an excellent base for such an approach, but very few have been made or even started. While addressed to automatic control, the approach used to develop the preliminary master plan for combined sewer overflow control in San Francisco(11) is an excellent example of the basic approach recommended. Modeling, in the traditional hydrological sense, is of extremely limited value in the total metropolitan overview. While zero-pollution was a concept that was not even given serious consideration not too long ago, our new national policy has defined this as a goal. Therefore, all metropolitan-wide alternatives should consider this goal as the upper limit of objectives in developing any plans, together with partial-accomplishment stages that should be adequately flexible to permit approaching the ultimate solution over a series of compatible phases of development.

For estimates of total rainfall-total runoff relations, the results obtained for Gray Haven, Northwood, Oakdale and Boneyard Creek could be very helpful.(129) These could be generated very simply using a local Weather Service gage record. A possible extension of this information would be in obtaining rainfall excess for unit hydrograph applications.

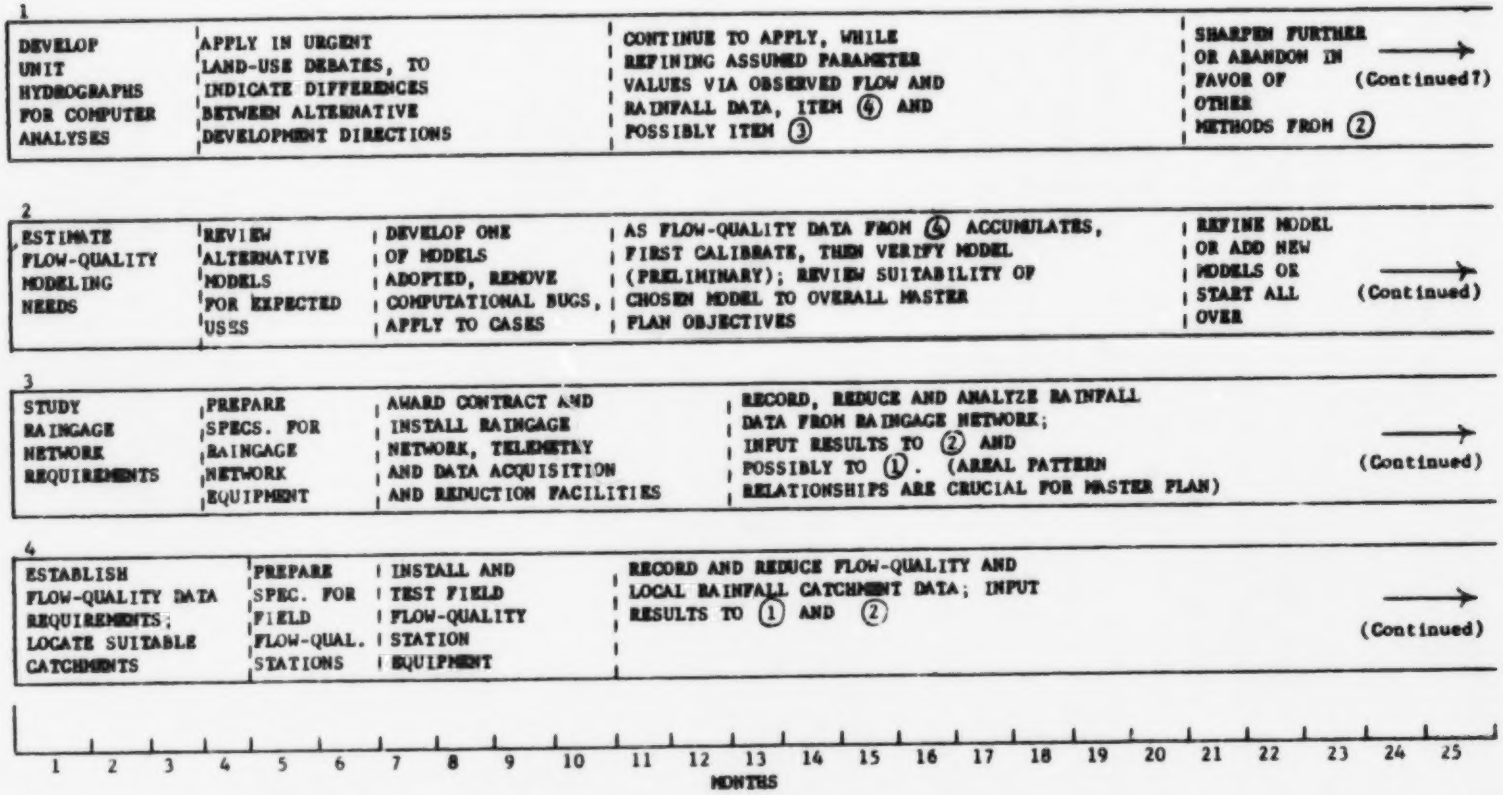
The writer was asked, by the City Engineer of a large community that was about to initiate a master plan for combined sewer overflow abatement, for suggestions on the development of a suitable basic data base for meeting immediate subcatchment planning needs and for long-range master planning. Recommendations are summarized in Figure 4, divided into four parts.

Item 3 in Figure 1, raingage network, refers to the advanced type San Francisco, where (as noted above) signals from thirty raingages (and) over a hundred water-level gages) are transmitted as frequently as every 15-seconds to a central data acquisition and recording station as a part of a system developed for the City.(11,105,130) A computer program ("SYMAP"), operated off-line, interpolates between raingage network readings and prints out rainfall depths for a fine grid covering the entire City of San Francisco.

The main reason for urging early installation for the raingage network, Figure 4, was to permit ascertaining as soon as possible the predominant spatial variations of rainfall peculiar to that metropolitan area, whatever they might be. At present, there is only one first-order Weather Service gage in that particular entire metropolis.

Items 1, 2 and 4 have been discussed earlier in this paper, but to reiterate somewhat, of crucial importance is the fact that entirely different models may be needed for assessing alternative land-use plans, for near-term replacement of the rational method for design, for a preliminary master plan, for implementing a master plan, and for other purposes that occur to a department as it proceeds to embrance all of its responsibilities in a comprehensive framework.

The City Engineer was advised that the writer suspected he would wish to include, at some time, waterquality parameters (perhaps both concentrations and event amounts), that peak flows will not be nearly as important as occurrence volumes in some applications, and because not more than one element can be maximized or minimized or optimized in any given calculation, either a not now available very flexibile model or a group of different models will be required. Further, because new storage will be required for almost any overflow-capture scheme, a long-term objective would be acquisition and mastery of models that would admit that feature for any point or points in a catchment.



1
DEVELOP UNIT HYDROGRAPHS FOR COMPUTER ANALYSES
APPLY IN URGENT LAND-USE DEBATES, TO INDICATE DIFFERENCES BETWEEN ALTERNATIVE DEVELOPMENT DIRECTIONS
CONTINUE TO APPLY, WHILE REFINING ASSUMED PARAMETER VALUES VIA OBSERVED FLOW AND RAINFALL DATA, ITEM ④ AND POSSIBLY ITEM ③
SHARPEN FURTHER OR ABANDON IN FAVOR OF OTHER METHODS FROM ②
(Continued?)
2
ESTIMATE FLOW-QUALITY MODELING NEEDS
REVIEW ALTERNATIVE MODELS FOR EXPECTED USES
DEVELOP ONE OF MODELS ADOPTED, REMOVE COMPUTATIONAL BUGS, APPLY TO CASES
AS FLOW-QUALITY DATA FROM ④ ACCUMULATES, FIRST CALIBRATE, THEN VERIFY MODEL (PRELIMINARY); REVIEW SUITABILITY OF CHOSEN MODEL TO OVERALL MASTER PLAN OBJECTIVES
REFINE MODEL OR ADD NEW MODELS OR START ALL OVER
(Continued)
3
STUDY RAINGAGE NETWORK REQUIREMENTS
PREPARE SPECS. FOR RAINGAGE NETWORK EQUIPMENT
AWARD CONTRACT AND INSTALL RAINGAGE NETWORK, TELEMETRY AND DATA ACQUISITION AND REDUCTION FACILITIES
RECORD, REDUCE AND ANALYZE RAINFALL DATA FROM RAINGAGE NETWORK; INPUT RESULTS TO ② AND POSSIBLY TO ①. (AREAL PATTERN RELATIONSHIPS ARE CRUCIAL FOR MASTER PLAN)
(Continued)
4
ESTABLISH FLOW-QUALITY DATA REQUIREMENTS; LOCATE SUITABLE CATCHMENTS
PREPARE SPEC. FOR FIELD FLOW-QUAL. STATIONS
INSTALL AND TEST FIELD FLOW-QUALITY STATION EQUIPMENT
RECORD AND REDUCE FLOW-QUALITY AND LOCAL RAINFALL CATCHMENT DATA; INPUT RESULTS TO ① AND ②
(Continued)
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MONTHS

FIGURE 4 A SUGGESTED BASIC DATA BASE FOR MEETING IMMEDIATE SUBCATCHMENT PLANNING NEEDS AND FOR DEVELOPMENT OF A MASTER PLAN

For the preliminary master plan evaluation, it was suggested that the department consider seeking access to the simple but excellent rainfall-runoff-quality model developed recently for the Corps of Engineers.(31)

Lastly, it was argued that for inputs for all models nothing short of an approximation of the full rainfall record should be used. That is, around 20-years of U.S. Weather Service "excessive precipitation" data for the City at 5-minute intervals (shortest period available) should be routed through the model used, each time it is used, until some indication of the use of a lesser, selected set of storm data becomes apparent. It was pointed out that the City Engineer would be hard-put to explain an artificial, synthetic storm's frequency to irate citizens who have been flooded or to a State official regulating overflows. Defense against storms of record is rather direct, and in the writer's opinion the only realistic option open to a public official. The temptation to use synthetic, phoney, artificial input data should be resisted. The credibility of an actual Weather Service gage record is far greater in a practical sense than confections such as a "design storm".

SUMMARY AND CONCLUSION

For preliminary analysis and design of specific projects, the use of the simplest models is recommended because of the tremendous detail otherwise required and because of the dearth of calibration-verification data available for more sophisticated models. Rather than embark on a completely empirical adventure, it would be prudent to make ad hoc temporary rainfall-runoff-quality measurements, to calibrate and verify the crude models used. If actual field data is available, the use of detailed modeling techniques would be justified, but only to give a better insight on the underlying processes, and to develop a more rational understanding of the performance of the simpler models used, although opportunities for design application may arise. The guiding rule should be a compatibility of tool sophistication with scale of use and quality of input data. Artificialities, such as the intensity-duration-frequency curves used in the "rational method", should be avoided and historical storm data should be used as inputs for all types of models.

All but the rational method require use of a computer for efficient calculation. This point has been reserved until last in order to avoid overemphasizing the role of computers.

The importance of drainage design is implied by some findings of the National Water Commission: the estimated replacement value of existing storm and combined sewer systems alone is $36-billion; and the collective costs of urban facilities over the past have represented seven-tenths of total national expenditures for all water resource facilities, and this proportion is expected to increase considerably in the future.(132) When and if abatement of pollution from urban storm runoff becomes a national reaility, these investments will be raised almost astronomically.

In closing, it appears appropriate to refer the reader to a recent bibliography, with abstracts, covering over six-hundred papers dealing with the subject of urban hydrology.(133) Lastly, use of unit hydrographs by the Corps of Engineers for estimating flows from local drainage in conjunction with river protection works is illustrated in the proceedings of a 1970 seminar.(134)

ACKNOWLEDGMENTS

The contents of this section of the paper have drawn heavily on an ASCE Program report(109) and a recent co-authored paper.(135) Preparation of both of these and the present paper was supported by means of a contract between ASCE and the Office of Water Resources Research, U. S. Department of the Interior.

POSTSCRIPT

There are supplementary reports to Footnote 25(136) and Footnote 26.(137) Features of both the San Francisco and Chicago master plans are outlined in a recent ASCE Program Technical Memorandum.(138) Two other recent issues are relevant to the subject of urban streamflows.(33,139)

SOME MANAGEMENT POSSIBILITIES

In the past, drainage conduits were deliberately designed to accelerate the movement of collected storm water to receiving water bodies via gravity flow, and seldom was detention storage incorporated in systems because of the preoccupation with rapid surface water removal. However, in-system storage is being provided in a number of new systems and is being added to some existing systems. Interest in in-system storage has come about because of broader integrated facilities-planning attitudes. As noted earlier, there is widespread interest in multi-purpose drainage facilities that exploit opportunities for provision of add-on water-based recreation, substantial increases in levels of flooding protection for buildings at small increments of cost, use and reuse of storm water for water supply via artificial groundwater recharge and related means, and reduced and/or programmed pollution burdens at outfalls. These all require utilization of storage.

Requirements for assessment of storm drainage flooding damage have been studied, and alternatives to the direct disposal of stormwater runoff have been explored.(141)

Peak flows of storm drainage facilities can be attenuated by a number of structural and land-use control means, such as by the use of designed ponding(142) and encouragement of land development site grading patterns that will increase flow distances over unpaved areas.(143) The turf areas separating pavements have long been exploited to produce ponding of overland flow, with consequent savings in drain size and reduction of peak outflows, at airports,(144) highway interchanges(145) and for shopping centers.(146) Also, recreational areas have been utilized for temporary detention storage.(147)

Storm drainage damages are the consequence of storing water in the wrong places, such as in the basements of homes. Hence, greater use of designed storage in lieu of undersirable storage depends for justification on relative protection from flooding afforded by different system plans at corresponding differences in cost. That is, trade-offs between benefits and disbenefits should be resolved by choosing among various mixes of flow-acceleration and storage components. Unfortunately, the contemporary absence of a satisfactory body of hydrologic and economic field data on urban storm drainage system floods constitutes a monumental liability in the assessment of those floods and their associated damages.

The cost of alleviating pollution from combined sewer overflows, by means other than separation into independent storm and wastewater systems, would be greater than the combined amount required to eliminate existing deficiencies in wastewater treatment facilities and to provide new treatment capacities to meet replacement and population growth needs between 1971 and 1974.(148) Because twice as many people are served by separate storm sewer systems as combined sewer systems, it is evident that the cost of alleviating pollution from stormwater discharges would be even much more substantial. Stated another way, the cost of alleviating pollution from combined sewer overflows and storm sewer discharges would be at least twice the replacement value of all such existing sewers.

The U. S. Government has been involved for several years in the development of measures for countering pollution from combined sewer overflows.(149,150) The ultimate solution for the problem of pollution abatement from both urban storm sewer discharges and combined sewer overflows is the treatment of such flows prior to their release into receiving waters. Capturing, transporting and treating all discharges/overflows at unattenuated flow rates would require gigantic collection sewers, pumping stations and treatment facilities - all of which would be used a very few hours per year. Therefore, practically all schemes for systemwide discharge/overflow collection and treatment incorporate some form of auxiliary storage for the purpose of attenuating sudden collection and treatment facilities inflows, with the objective of scaling down the size of such facilities to reasonable and manageable proportions.

On the basis of limited indications, it appeared that complete capture of of combined sewer stormwater flows would require perhaps 300-m^3 of storage per hectare of drainage area.(151) However, more recent estimates place storage requirements nearer to about 600-m^3/ha. The amount of storage required can be reduced by taking advantage of the fact that, because of the areal and temporal varability of rainfall intensity, the proportion of main sewer capacity in use at a given time varies between various main sewers throughout a rainstorm.

Of three basic approaches involved in the most advanced comprehensive attacks on the combined sewer overflow problem, one utilizes exploitation of ambient storage in existing trunk sewers by manipulating add-on constrictions (movable dams or gates) in outfall sewers. The constrictions are placed in the vicinity of existing regulators, and all flows other than from rare rainfalls are released thereby at attenuated rates to existing interceptors and thence to existing treatment facilities. That is, by raising the constructions in those main sewers where the potential storage volume is underused at a given time during a storm, and lowering the constrictions in advance of local inundating flow inrushes, the flows into the interceptors can be modulated for more efficient use of the interceptors, reflected in a greater diversion of flows to the treatment plant via the interceptors and consequent reduced overflows to the receiving waters. This basic approach is essentiallv exemplified at Minneapolis-St. Paul,(152-156) Detroit,(157-160) Seattle,(161-164) and Cleveland.(165)

A second basic approach incorporates new storage located at elevations well below all street sewers, in the form of tunnels or vaults, and new or auxiliary discharge/overflow treatment works. It might or might not have ancillary features such as pretreatment basins or pumped-storage power generation to offset the substantial energy requirements for eventual lifting of flood waters from underground storage to the ground surface. A bonus of this approach is that street trunk sewers would be converted into manifolds of diversion diffusers, with the result that their effective hydraulic capacity would be raised. This basic approach is exemplified in the pioneering plan developed in Chicago(166,170) and in analagous plan (113,114) alternatives under consideration elsewhere.(171)

The third basic approach is embodied in a plan for San Francisco(1i) which includes: a single, new combined-flow treatment plant; a number of new detention reservoirs located immediately below the streets in the upstream portions of a majority of catchments; a number of new shore-line detention reservoirs; a deep cross-system storage and transmission tunnel; and achievement of a total system fully automatic operational control capability.

All three basic approaches incorporate plans for some degree of automatic control because of their complexity.(106)

Because combined sewer overflows occur very suddenly, any facilities provided for treatment of potential overflows must be put on line almost instantaneously. This means that they would have to be activated immediately with the occurrence of any stormwater flow that would exceed interceptor sewer capacity. Further, such plants might be idle more than about nine-tenths of the year. Effectiveness of overflow pollution abatement using treatment facilities designed specifically for that purpose therefore will require some form of automatic operational control. Remote supervisory control would quite likely not be adequately responsive. The control logic required has yet to be developed, and it is possible that different metropolitan sewer systems will require their own fairly unique logic development.

Pollution abatement of storm water from separate systems of storm drains by means of treatment faces very nearly the same difficulties. In one sense, requirements are more severe because all storm water must pass through new and special treatment facilities, there being no interceptor sewers in such systems to divert small storm occurrence flows to perennially operated wastewater treatment plants.

Consideration may be given to the incorporation of stormwater conduits in multiple-service tunnels. There are numerous existing utility tunnel systems in Europe, Asia and North America.(172) Most utility tunnel concepts(1972) deal with structures located fairly close to the ground surface, and include possibilities for limited incorporated of storm sewers.(174) As an adjunct or alternative, consideration could be given to much deeper locations, at least for main feeders and arteries, for power transportation, surface water and wastewater, and for other services.(175)

Lastly, possibilities exist in rainfall-deficient regions for deliberate capture of urban storm water to augment water supplies, provided the quality is suitable, a requirement that might be met by various managerial schemes.(176) Manipulated recharge of groundwater supplies with surplus surface water has been practiced for some time in some parts of such regions.(177,178)

RESEARCH STATUS AND NEEDS

"The field of urban hydrology is almost devoid of modern research investment."(179) "More research is needed to develop understanding of the whole stormwater pollution problem. This research should cover the hydrology the hydraulics, the treatment, the effects on receiving waters, and related factors."(180) ".... too few data have been collected to describe the effect of urban and suburban development on flood runoff."(16) As a consequence, little is known about the rainfall-runoff process and still less about rainfall-runoff-quality, particularly for sewered catchments. ".... considering the huge relative investment in sewered systems and the almost complete absence of hydrologic data on sewered systems, it has been obvious for quite some time that this is the sector most woefully needing research attention."(181) Suitable data collected with properly coordinated instrumentation in networks representing a variety of climatic, topographic and land-use conditions are virtually non-existent. Further, while all metropolitan areas have underground systems of storm drainage, flooding of waterways is a serious problem in some metropolises but negligible or non-existent in others. Additionally, flood amelioration is preoccupied with rainfall-runoff whereas rainfall-runoff-quality processes are of more vital concern nationally.

A national program for acquisition of needed sewered catchment data has been proposed together with a documentation of its justification,(1) but progress in implementation has been painfully slow. In anticipation of the possible future availability of new data, considerations for modeling sewered urban catchment rainfall-runoff-quality processes were detailed(182) and considerations for characterizing rainfall time and spatial distributions in future research were explored.(183) In support of the latter, existing raingage networks were surveyed.(184,185) The U.S. Geological Survey is developing a program of data collection and studies to serve national needs in urban hydrology.(186,187) The Office of Water Resources Research has developed a broad program of projected urban water resources research,(89) including urban hydrology analyses.

The procedure used for design of storm sewers in the United States is almost exclusively the "rational method."(110) The method yields only an estimated peak flow but none of the other attributes of a hydrograph. Its many limitations have been reviewed elsewhere.(101) A full hydrograph is needed for the design of detention storage, for evaluation of pollutant burdens, for design of stormwater pollution abatement facilities, for designing local protection works along streams, such as pumping stations for passing local drainage flows over levees and dikes, and as inputs for design of stream and river development works. Also, quantification of the effects of urbanization on the hydrologic regimen is dependent in many cases upon the availability of sewer outlet hydrographs. Further, as urban water management problems become increasingly acute, the need for multiple-use of water becomes more evident. In exchanging one use for another, for example by using storm water as a source of water supply, knowledge of the time-histories of flows and water qualities is essential for reliable design of transfer facilities.

By 1969, only a very few sewered catchments had been gaged; records seldom extended over more than a very few seasons; although rainfall-runoff had been measured, for still fewer of the catchments had rainfall-runoff-quality measurements been made; runoff and runoff-quality measurements had been made at outfalls but not in-system; and data reliability was often suspect because of known instrument errors, lack of observations synchronization, and too crude signal resolution.(106) Proportionally more rainfall-runoff data was available for partially sewered areas(190) but the amount was nevertheless pitifully small. Since then, field research sponsored by the Environmental Protection Agency(149) and others has extended the data base somewhat.(140) However, not much new data has been acquired on completely sewered catchments using flumes or weirs rather than stage-gages for determining discharge.

All hydrologic models require the application of some degree of subjective judgment in the assignment of numerical values to model component parameters. Reliability of judgment is improved when parameter values are based upon fits to observed events for the catchment being modeled, and is maximized when parameter values can be generalized for a number of catchments in terms of physical features such as degree of imperviousness, type of land use and channel density. The usual purpose of modeling is to project performance for future events. For urban catchments, this usually involves physical features such as degree of imperviousness, type of land use and channel density. The usual purpose of modeling is to project performance for future events. For urban catchments, this usually involves physical changes over time such as revised land use, new detention storage and modified land-management practices. Independently from any consideration of catchment changes, recorded precipitation must be adapted in some way to represent expected future precipitation. Thus, projections of runoff-quality events inevitably require the application of some degree of judgment. The principal reason for testing or calibrating a model with observed data is to enhance confidence in its use, and this approach of going from the present to the future is employed in all water resource computations involving risk and uncertainty.

On top of all of the above is the inherent difficulty with any runoff model in the necessarily subjective separation of abstractions (infiltration, depression storage, etc.) from total rainfall to resolve rainfall excess (amount and pattern), which is the input from which an equal volume of direct

runoff is generated by models of one kind or another.
Nine projects were identified in 1970 where the urban rainfall-runoff process was being modeled for sewered and partially sewered catchments(190) A project completed since(115-117) has developed the only existing model that readily accommodates water quality parameters. The model was developed as an assessment technique for comparing alternative solutions by means of a comprehensive computer program capable of "representing urban stormwater runoff phenomena, both quantity and quality, from the onset of precipitation on the basin, through collection, conveyance (both combined and separate systems), storage, and treatment systems to points downstream from outfalls which are significantly affected by storm discharges."(191) An improved version of the model has been tested for a 1500-ha. San Francisco(191) sewered catchment.(118) These and any other models purportedly developed for sewer application suffer substantially from a severe shortage of data for their calibration, verification and, particularly, for their realistic application to non-gaged catchments.

In default of suitable data, some astute planners have applied necessarily weakly-founded generalized unit hydrograph parameters(107,108,229)in analysis of areawide storm sewer performance. The Soil Conservation Service has developed parameters for synthetic triangular unit hydrographs applicable to small non-urban catchments throughout the country.(192) Local synthetic triangular unit hydrograph parameters have been developed for the San Francisco Bay Region using observed data for non-urbanized catchments and empirical indications for urbanized catchments for which suitable data were nonexistent.(193) A unit hydrograph has benn defined for use in drainage planning for the Denver metropolitan area, Colorado.(110,109,33) Use of unit hydrographs by the Corps of Engineers for estimating flows from local drainage in conjunction with river protection works is illustrated in the proceedings of a 1970 seminar.(194)

A sewered catchment runoff generation technique developed in the United Kingdom has been tested using some U.S. data.(140,112)

A dearth of historic runoff records on urban streams, compounded by the rapid changes that have taken place in urban watersheds, has thwarted effective use of existing simulation models for planning.

A mathematical watershed simulation model that operates with continuous, multi-seasonal hydrologic data was used in a study of the effects of rainfall data time-interval and raingage density on simulated hydrographs compared with observed hydrographs for four sewered catchments.(113) Experimentation was severely hampered by the almost total absence of appropriate data.

An empirical total rainfall-total runoff relation has been developed that approximates the runoff volumes measured at the outlets of four sewered urban catchments.(129)

A recently developed model requires as computational inputs: the ratio of direct runoff volume to the volume of rainfall subsequent to the beginning of direct runoff; and the associated duration of rainfall excess.(195) For predictive application these two factors must be guessed where no runoff records are available or generalized in some way in the rather unique instances where some runoff data exist.

An elegant mathematical model has been developed for routing intercepted stormwater flows in an interceptor sewer of a combined system,(196) and was verified in part using unsteady flow laboratory test findings.(197)

ACKNOWLEDGEMENT

This material was taken from a chapter prepared for the IHD and supported by means of a contract between the American Society of Civil Engineers and the Office of Water Resources Research, U. S. Department of the Interior. The first draft of this chapter, distributed for U.S. review, was also supported entirely by OWRR.(100) Supplementary considerations were reported in a later paper.(198)

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QUANTITY ASPECTS OF URBAN STORMWATER RUNOFF

By
William H. Espey *
David E. Winslow

1.0 RAINFALL/RUNOFF

In order to properly design urban drainage facilities, consideration must be given to the required design storm rainfall, the rainfall-runoff relationships as affected by the surface and geometric characteristics of the watershed. In general, the design of an urban storm drainage system should be guided by the following six criteria**:

1) The system must adequately dispose of all surface runoff, resulting from the selected design storm, without causing serious damage to physical facilities or serious interruption of normal traffic.
2) Runoff resulting from storms exceeding the design storm must be disposed of with the least amount of damage to physical facilities and interruption of normal traffic.
3) The storm drainage system must have a maximum reliability of operation.
4) The construction costs of the system must be reasonable with relationship to the importance of the facilities it protects.
5) The storm drainage system must require minimum maintenance.
6) The storm drainage system must be adaptable to future expansion with minimum additional cost.

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*President, Manager, respectively, Espey, Huston & Associates, Inc., Austin, TX.

**Master Drainage Manual - City of Austin, 1974, by URS/Forrest & Cotton, Inc., and Espey, Huston & Associates, Inc.

1.1 The Concept of Rainfall Excess

The two parts of the runoff process have been called the land phase and the channel phase. This section discusses the land phase problem or methods of determining rainfall excess from observed basin rainfall given information about basin characteristics and previous rainfalls. There are many problems involved in successfully analyzing each part of the runoff process; however, the land phase problem is possibly the more difficult process to analyze.

1.1.1 Relation Between Precipitation and Runoff

- According to Chow (1964), there are two major groups of interrelated factors which influence the runoff process:

a) Climatic factors

Precipitation: Form, type, intensity, duration, time distribution, areal distribution, frequency of occurrence, direction of storm movement, antecedent precipitation, and soil moisture.

Interception: Vegetation species, composition, age, and density of stands, season of year.

Evaporation: Temperature, wind, atmospheric pressure, nature and shape of evaporative surface.

Transpiration: Temperature, solar radiation, wind, humidity, soil moisture, kinds of vegetation.

b) Physiographic factors

Basin characteristics: Size, shape, slope, orientation, elevation and stream density, land use and cover, surface infiltration condition, soil type, geological and topographical conditions.

Channel characteristics: Size and shape of channel cross-section, slope, roughness, and length.

1.2 Review of Methods for Determining Rainfall Excess

This section reviews some of the recent technology which is available for modeling land phase problems for the purpose of rainfall excess prediction. In particular, infiltration models, API-type models, and functional rainfall models will be discussed.

1.2.1 Infiltration Models - The principle of the infiltration approach is straightforward, i.e. runoff occurs when the precipitation intensity exceeds the rate of infiltration. A simple, empirical method based on the infiltration concept described by Linsley et. al. (1949) is the so-called Φ-index method. This method, which ignores time and space variation of infiltration phenomena, is a simple expression for the infiltration capacity of the basin as shown in Fig. 1. If a time-intensity graph of rainfall is constructed, the Φ-index is the average rainfall intensity above which the volume of rainfall equals the volume of observed runoff. Thus Φ is the constant rate at which water must be extracted from the rainfall to yield the observed runoff. In some applications the Φ-index is related to an antecedent precipitation parameter or to season of the year in order to improve results.

Obviously, the Φ-index method is a simplified model for rainfall excess determination since it ignores temporal and spatial variation of infiltration rates. However the method may yield acceptable results in some cases, particularly where homogeneous conditions exist on the watershed.

For example, Sauer (1970) after examining other methods (to be discussed below) used the Φ-index method and obtained useable results. Fig. 2 from Sauer shows the median Φ-index, and standard error bands for each month of the year based on about 500 storms from 21 basins in Northern Louisiana. The use of such a relation is justified in this case since mean basin rainfall determination is poor due to inadequate rain-gage coverage.

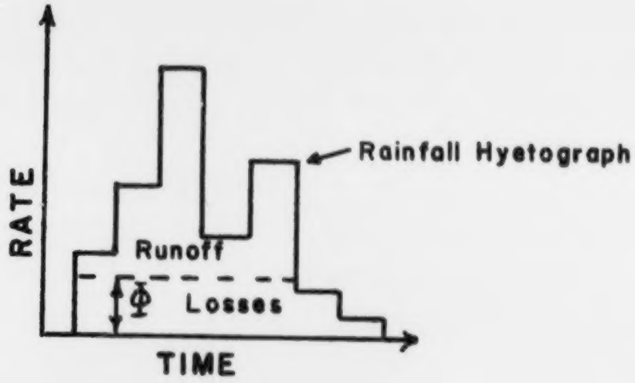


Fig. 1 Schematic of Φ-Index
(after Linsley *et*. *al*., 1949)

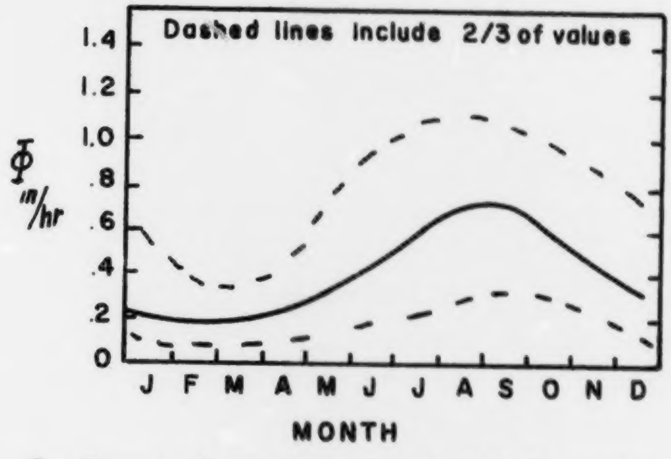


Fig. 2 Seasonal Variation of Infiltration Index, Φ
(after Sauer, 1970)

Grayman and Eagleson (1969) developed a model of catchment dynamics using a variation of the Φ-index method to determine rainfall excess. These authors reason that only a fraction, C, of the basin rainfall will result in runoff. The parameter, C, represents the net effect of the interaction of the complicated factors described above in Section 1.1, that is $i_e = Ci$ where i_e is total rainfall excess and i is total basin rainfall. The percentage of rainfall that appears as runoff will generally increase during wet periods due to saturation of the soil and other factors; therefore, C is modeled as an exponential decay function,

$$C = 1 - \exp\,(B_4 AP30) \tag{1}$$

where B_4 is a curve-fitting parameter which reflects the influence of the soil-vegetative cover and AP30 is a 30-day total antecedent precipitation.

Horton (1939) found from runoff plot experiments that the soil infiltration capacity rate as a function of time decreases during the infiltration process by the relation,

$$\frac{df}{dt} = -k(f-f_c) \tag{2}$$

where f is the actual infiltration capacity, f_c the minimum infiltration capacity which is reached if the infiltration process continues indefinitely, and k is a constant. A solution to this differential equation yields the well known Horton formula,

$$f = f_c + (f_o-f_c)e^{-kt} \tag{3}$$

where f_o is the initial infiltration capacity at the beginning of the storm. A representation of equation (3) is shown in Fig. 3.

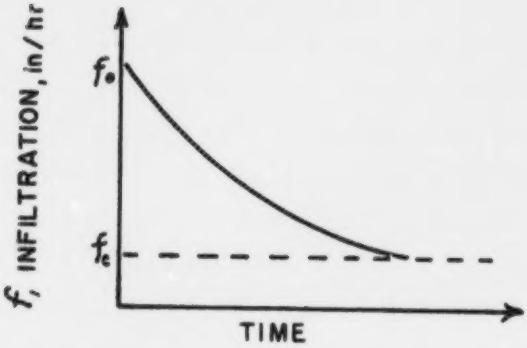


Fig. 3 Horton Infiltration Formula

Three drawbacks of the Horton formula have been identified: (1) The formula assumes an adequate and continuous supply of water available for infiltration. Thus, a model based on the Horton formula is only approximately true for storms where the precipitation rate drops below the infiltration capacity rate. (2) The method requires an estimate of f_o at the beginning of the storm. This requirement is the main difficulty of the method although estimates of f_o have been made using correlation techniques. (3) The rainfall excess value determined using equation (3) refers only to overland flow.

In spite of the above drawbacks, the Horton formula allows a reasonable representation of the distribution of rainfall excess during a storm period and there are numerous models based on the Horton formula. Betson (1964) used the formula to develop a model for a watershed infiltration capacity function. The EPA Storm Water Management Model (Metcalf and Eddy, Inc.,et al (1971)) uses this formula to determine soil infiltration losses.

If the area under the Horton infiltration capacity curve for time t is considered as a volumetric loss equal to $\int_0^D (c+be^{-nt})d$ where D is storm duration, c, b, and n are coefficients which correspond to f_c, (f_o-f_c) and k respectively; then, this volumetric loss, L should equal rainfall, R, minus runoff, RO, or,

$$L = R-RO = \int_0^D (c+be^{-nt})dt \qquad (4)$$

Because for a given storm, the initial infiltration capacity may not be the maximum value, the integration of equation (4) should be over an interval t(sm)-(D+t(sm)) as shown in Fig. 4 where t(sm) is the time which corresponds to an initial capacity rate, f(sm).

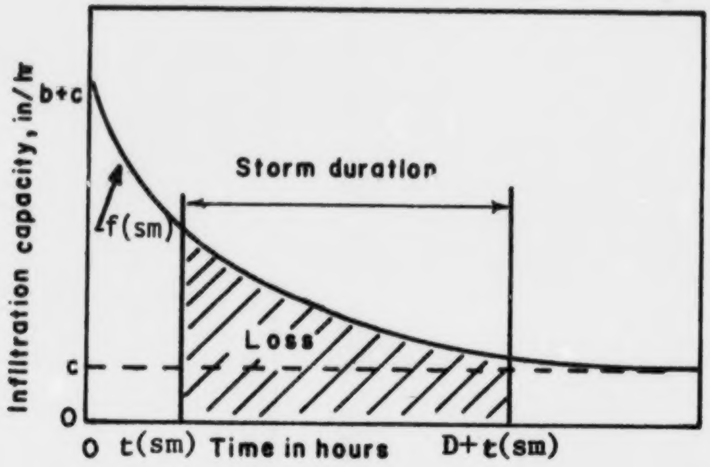


Fig. 4 Graphical Representation Of Infiltration Capacity Function

This initial rate may then be related to a soil moisture index, sm. After additional modification of the basic Horton formula, Betson (1964) integrates the formula to obtain,

$$L = ct-(b/n)e^{-nt} \Big|_{t(sm)}^{D+t(sm)} \qquad (5)$$

When equation (4) was fitted to watershed data using regression methods, a useable expression for determining rainfall excess resulted. Shown in Fig. 5 are infiltration curves developed by Horner and Jens (1942) based on studies in St. Louis. The upper curve was assumed for residential areas with soils having high infiltration capacities.

In a recent application (EPA/Rice University Grant No. 802433) of the SWMM to a small urban watershed in Houston, Texas and to The Woodlands New Town development near Houston, Texas the Horton infiltration coefficients were adjusted based on U. S. Geological Survey (USGS) rainfall-runoff data in the Houston area. This was

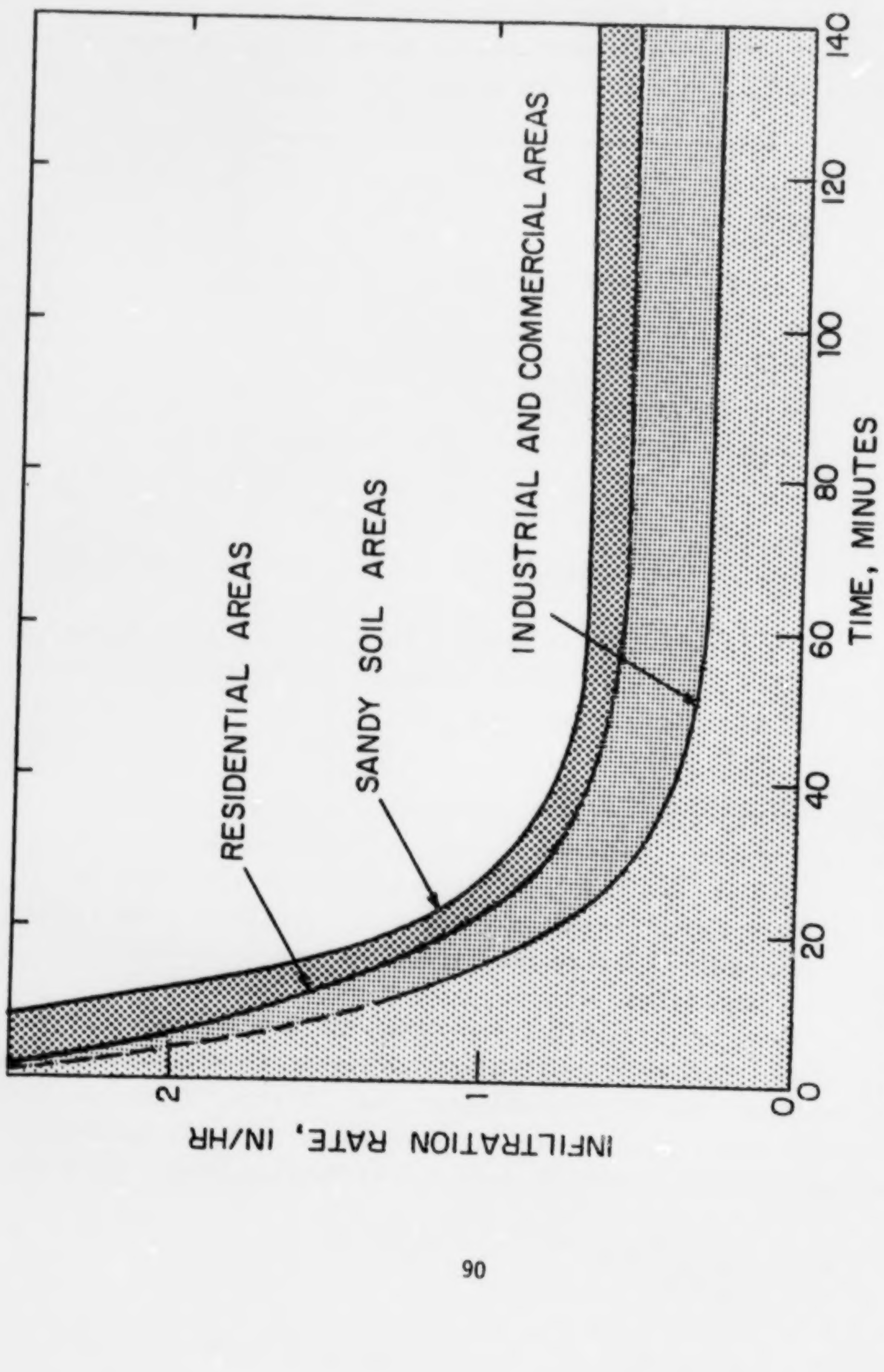


FIGURE 5 PERVIOUS SURFACE INFILTRATION-CAPACITY CURVES.(3)
(After Horner and Jens, 1942)

accomplished by calculating "effective infiltration rates" for a number of observed storm events and plotting these values versus storm duration in order to determine graphically the coefficients for Horton's equation. It was found that the initial infiltration rate, f_o, varied from 0.75 to 3.0 inches per hour depending on antecedent soil moisture conditions. Further, 0.12 inches per hour was found to be an acceptable final infiltration rate, f_c, while the decay rate, k was found to be .00005 sec^{-1}.

1.2.2 API-Type Models - Several types of models can be generally classed as API-type models. In this section, some of these models are considered including graphical coaxial models and regression models.

1.2.2.1 Graphical and Numerical Coaxial Models - Kohler and Linsley (1951) describe a graphical coaxial model for determining direct runoff volume; such a relation is shown schematically in Fig. 6. The first requirement for use of such a model is the

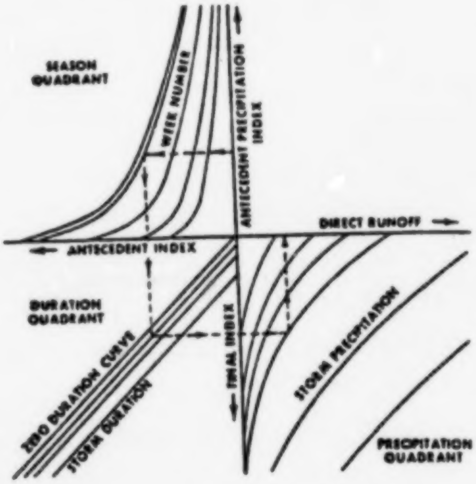

SEASON QUADRANT
WEEK NUMBER
ANTECEDENT PRECIPITATION INDEX
DIRECT RUNOFF
ANTECEDENT INDEX
DURATION QUADRANT
ZERO DURATION CURVE
STORM DURATION
FINAL INDEX
STORM PRECIPITATION
PRECIPITATION QUADRANT

Fig. 6 Standard API Type Rainfall-Runoff Relation

determination of an index of the moisture condition of the drainage basin at the moment precipitation starts. The most common index with regard to this initial moisture condition is the antecedent precipitation index API.

The API may be defined in many different ways. For exampl Grayman and Eagleson (1969) defined it as the total precipitation over a preceding N-day period. Another API model has the formula

$$API = P_o+bP_1+b^2P_2+ \ldots +b^kP_k \tag{6}$$

where b^k is a constant less than unity which is a function of time. If a day by day value of the index is required, b^k may be assumed to decrease exponentially with time as $b^k = K^t$, where K is a recession constant. Thus

$$API_t = API_o \ K^t \tag{7}$$

where API_o and API_t are initial and t-day values of the index respectively. The value of the index after any day is related to the index the day before since $API_2 = K(API_1)$ where $t = 1$. According to Chow (1964), the value of K normally ranges between 0.85 and 0.98 and depends on potential evapotranspiration of the region as well as time of year.

In application, the API-type model shown in Fig. 6 is entered with an API value for the given season of the year; then (as indicated by the arrows in the figure) is adjusted with a storm duration value and finally is adjusted using a value of storm precipitation to obtain direct runoff. The model as given determines the total direct runoff for an event of any duration in terms of total precipitation. In practice, such a graphical model is developed by trial and error using observed data until good agreement between observed and predicted runoff occurs.

The relationships illustrated in Fig. 6 are strictly applicable to the storm event only; if continuous prediction of runoff for say 5-hour periods is required, some modification of the graphical API-type model must be made. Sittner et. al. (1969) made such a modification in order to construct a model useful for continuous flow forecasting.

1.2.2.2 Regression Models - Moore (1968) developed an API-type model using graphical linear regression methods, which is similar to a coaxial model. In the method, rainfall excess is related to mean basin rainfall, rainfall duration, API, and season of the year as in the U. S. Weather Bureau coaxial model. The API used by Moore is a simplified 6-day index whereby API = $6/6\ P_1 + 5/6P_2 + \ldots. + 1/6P_6$; other data for the model is obtained from observed rainfall and runoff records. In Moore's method, storm runoff is graphically related to the above parameters by the method of deviations of graphical multiple regression rather than by the coaxial method. Storm runoff is plotted against precipitation and deviations are then plotted against storm durations and the second deviations are plotted against API. The last plot has season of year as a parameter and the trial curve relating storm rainfall and runoff. A second trial curve of storm rainfall and runoff is then used and the process is repeated. After a few iterations of the process, acceptable results can be obtained. Moore (1968) used the method to obtain synthetic daily direct runoff estimates and obtained good results.

In order to evaluate the effects of increasing impervious cover on the runoff from a small urban watershed (Waller Creek--4.13 square miles), Espey et. al. (1965) developed a rainfall-runoff regression relationship which relates total runoff to impervious cover, antecedent-precipitation index, amount of rainfall, and duration of rainfall. Winslow and Espey (1972-73) developed a rainfall runoff regression model for The Woodlands a HUD New Town located near Houston, Texas. A relationship was obtained by multiple regression analysis of 142 storms for 21 Houston area watersheds. The following equation was obtained to predict the runoff for a given rainfall and watershed condition:

$$RUN = 0.325\ R^{1.23}\ M^{0.23}\ I^{0.067}\ SI^{-0.12} \tag{8}$$

where RUN is the total runoff in inches,
R is the total rainfall in inches,
M is the soil moisture index,
I is the percent impervious cover, and
SI is the soil index.

The data used to develop this relationship, except for the total runoff, were obtained from the USGS in Houston. The total runoff for each storm was obtained from the annual compilations of Houston urban watershed data from 1965-70, and from unpublished records of the Houston office of the USGS. The soil moisture index, M, is computed by essentially the same procedure as the antecedent precipitation index described by Linsley, et. al. (1958). The soil index, SI, is the maximum permeability of the soil in inches per hour.

1.2.3 Functional Models - A functional model proposed by the Soil Conservation Service and described by Kent (1968) is widely used. The relation between direct runoff and storm rainfall is determined by assuming an initial abstraction, I_a which includes surface storage, interception, and infiltration prior to the runoff event and a potential maximum retention, S. It is assumed that the ratio of actual retention $(P-I_a-Q)$ and potential maximum retention S equals the ratio of actual runoff, Q and potential maximum runoff $(P-I_a)$ as

$$\frac{P-I_a-Q}{S} = \frac{Q}{P-I_a} \tag{9}$$

where P and Q are accumulated rainfall (potential maximum runoff if $I_a = 0$) and accumulated direct runoff respectively. A rearrangement of equation (9) gives

$$Q = \frac{(P-I_a)^2}{(P-I_a)+S} \qquad (10)$$

which is the basic SCS rainfall-runoff equation. On the basis of experience, $I_a = 0.20S$ and therefore,

$$Q = \frac{(P-0.2S)^2}{P - 0.8S} \qquad (11)$$

Equation (11) was developed from experimental plots for numerous soils and vegetative cover conditions and is mainly applicable to small watersheds and for direct runoff from 24-hour storm rainfalls.

The value of S is related to the so-called runoff curve number according to the equation,

$$CN = \frac{1000}{10+S} \qquad (12)$$

Research data obtained by the Soil Conservation Service has provided a relation between CN's and various hydrologic soil-cover complexes for more than 3,000 soils. In addition, soils are divided into four hydrologic groups, depending on infiltration rate, and antecedent moisture is accounted for by division into three classes according to runoff potential. In practice, a composite CN for the watershed is used to enter Fig. 7, the rainfall-runoff chart which represents a graphical solution to equation 10.

Mills and Viesman (1972) developed empirical relationships based in part upon the Soil Conservation Service soil cover complex method to estimate the runoff from small watersheds. For rainfalls less than 1.5 inches, the runoff was predicted by a relationship between the percent impervious area on the watershed and the percent excess rainfall. An adjustment was made for initial abstraction combined with initial surface depression storage.

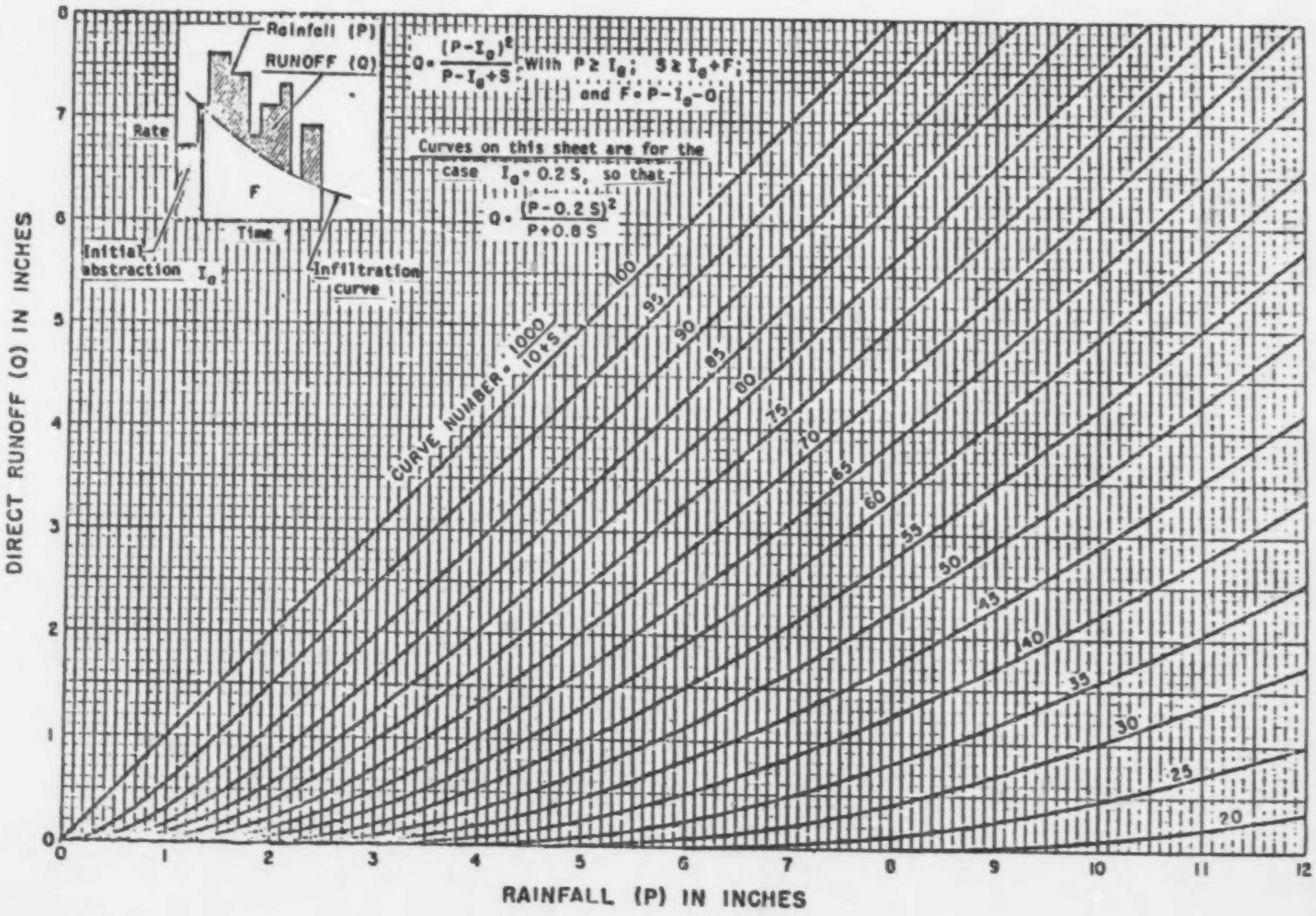


Fig. 7 --Solution of the runoff equation, $Q = \frac{(P - 0.2S)^2}{P + 0.8S}$ after Kent (1968)

or rainfall amounts greater than 1.5 inches, an additional increent of runoff was added due to the pervious areas by using hydroogic soil classes, vegetation cover, and other controlling arameters.

1.3 Overland Flow Analysis

One of the methods commonly used in estimating rates of runoff from small drainage areas such as air fields and urban impervious areas of uniform slope and surface characteristics is the hydraulics of overland flow. The following section will present development of the basic overland flow equations based on theoretical considerations and experimental data.

Two of the more important and classical studies of overland flow characteristics were by Horton (1935) and by Izzard (1946). Horton presented equations for overland flow based on turbulent flow considerations. Izzard's overland flow equations considered laminar flow conditions and were presented in the form of dimensionless hydrographs (Figure 8). Izzard's dimensionless hydrographs were verified in various laboratory tests and resulted in acceptable agreement between experimental and laboratory results. Horton's overland flow equations apply to a uniform rainfall supply of unlimited duration. In addition, the Horton equation for the recession portion of the overland flow hydrograph has no experimental verification. Izzard's dimensionless overland flow hydrograph is relatively simple and reflects non-uniform rainfall supply.

Equations of overland flow can be developed based on the assumption of laminar flow. If the depth of flow in the overland sheet is quite small, the quantity of water temporarily stored in this sheet surface attention is relatively large. For the conditions of laminar flow the following equation can be derived

$$\rho g(D-y)s = \mu \frac{dv}{dy} \tag{12a}$$

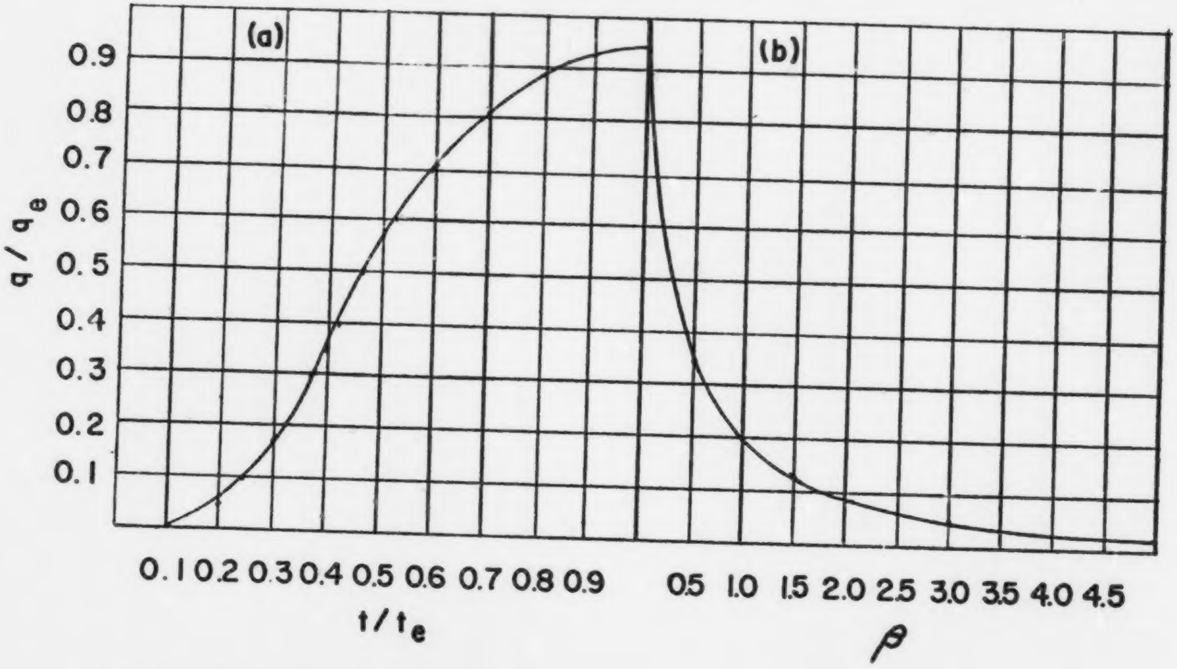


FIGURE 8. IZZARD'S DIMENSIONLESS HYDROGRAPH OF OVERLAND FLOW

where ρ is density, g is gravitational acceleration, and μ is absolute viscosity. The assumption is made that the slope, s, is so small that the sine and tangent are equal. Since μ/ρ is equal to kinematic viscosity ν,

$$dv = \frac{gs}{\nu}(D-y)dy \tag{13}$$

Integrating the above equation with the boundary conditions v = 0, y = 0

$$v = \frac{gs}{\nu}\left(yD-\frac{y^2}{2}\right) \tag{14}$$

Integrating from y = 0 to y = D and dividing by D, the mean velocity is

$$v_m = \frac{gsD^2}{3\nu} \tag{15}$$

and the discharge per unit width, q, equals vD or

$$q = bD^3 \tag{16}$$

where b is a coefficient involving slope and viscosity.

Based on Izzard's experiments the time to equilibrium can be expressed as a dimensionless overland flow hydrograph.

$$t_e = \frac{2Ve}{60q_e} \tag{17}$$

where t_e is defined as the time in minutes when flow is 97 percent of the supply rate and V_e is the volume of water in surface detention at equilibrium. From a strip of unit width the equilibrium flow q_e can be expressed as follows:

$$q_e = \frac{iL}{43,200} \tag{18}$$

where i is the rainfall rate (or the rate of rainfall excess if the surface is pervious) and L is the distance of overland flow.

The constant 43,200 gives q_e in cubic feet per second when i is in inches per hour. Substituting average depth (D_e) on the strip, where $D_e = V_e/L$, for outflow depth, Eq. 16 becomes

$$D_e = \frac{Ve}{L} = bq_e^{\ 1/3} \tag{19}$$

Substituting Eq. (18) for q_e gives

$$V_e = \frac{KL^{4/3} i^{1/3}}{35.1} \tag{20}$$

where V_e is the volume of detention (cu ft) on the strip at equilibrium. Izzard found experimentally that b could be expressed as follows:

$$b = \frac{0.0007i+c}{s^{1/3}} \tag{21}$$

where s is the surface slope and the retardance coefficient c is as given in Table I.

TABLE I

RETARDANCE COEFFICIENT c IN EQ. 21

Very smooth asphalt pavement..........	0.007
Tar and sand pavement.................	0.0075
Crushed-slate roofing paper...........	0.0082
Concrete..............................	0.012
Tar and gravel pavement...............	0.017
Closely clipped sod...................	0.046
Dense bluegrass turf..................	0.060

Izzard's overland-flow hydrograph in a dimensionless form is shown in Figure 8. A dimensionless recession curve defines the shape of the receding limb. At any time t_a after the end of rain, the factor β is

$$\beta = \frac{60 q_e t_a}{V_e} \tag{22}$$

where V_e is the detention given by Eq. 20.

1.4 Runoff Hydrograph

Water which constitutes the surface stream flow reaches the urban drainage system by various paths from the original point at which the rainfall first fell on the watershed. One of the principle paths of water entering the drainage system is overland flow. Overland flow is the flow across the surface of the watershed in which its flow properties are not dictated by the flow characteristics of the surface drainage system. In addition, other waters infiltrate through the soil and flow beneath the surface to the stream channels. The actual paths of movement of water to a surface drainage canal can be summarized as follows:

(1) Overland flow
(2) Inner flow
(3) Ground water flow

In urban watersheds the distance for overland flow is relatively short. The component of overland flow is an important element in the formation of the total runoff hydrograph. Inner flow or sub-surface flow represents the water which infiltrates the soil surface and moves laterally through upper soil strata and eventually enters the stream channel. The third element of runoff originates from precipitation which percolates into the water table. If the ground water table is penetrated by the stream channel, ground water flow may enter the stream channel. This flow has been characterized as base flow or dry weather flow.

Shown in Fig. 9 is a typical runoff hydrograph for an urban watershed. The hydrograph consists of a rising limb, crest stage, and falling limb or recession. On urban watersheds the rising limb is a function mainly of the characteristics of the storm, and the hydraulic conveyance characteristics of the improved

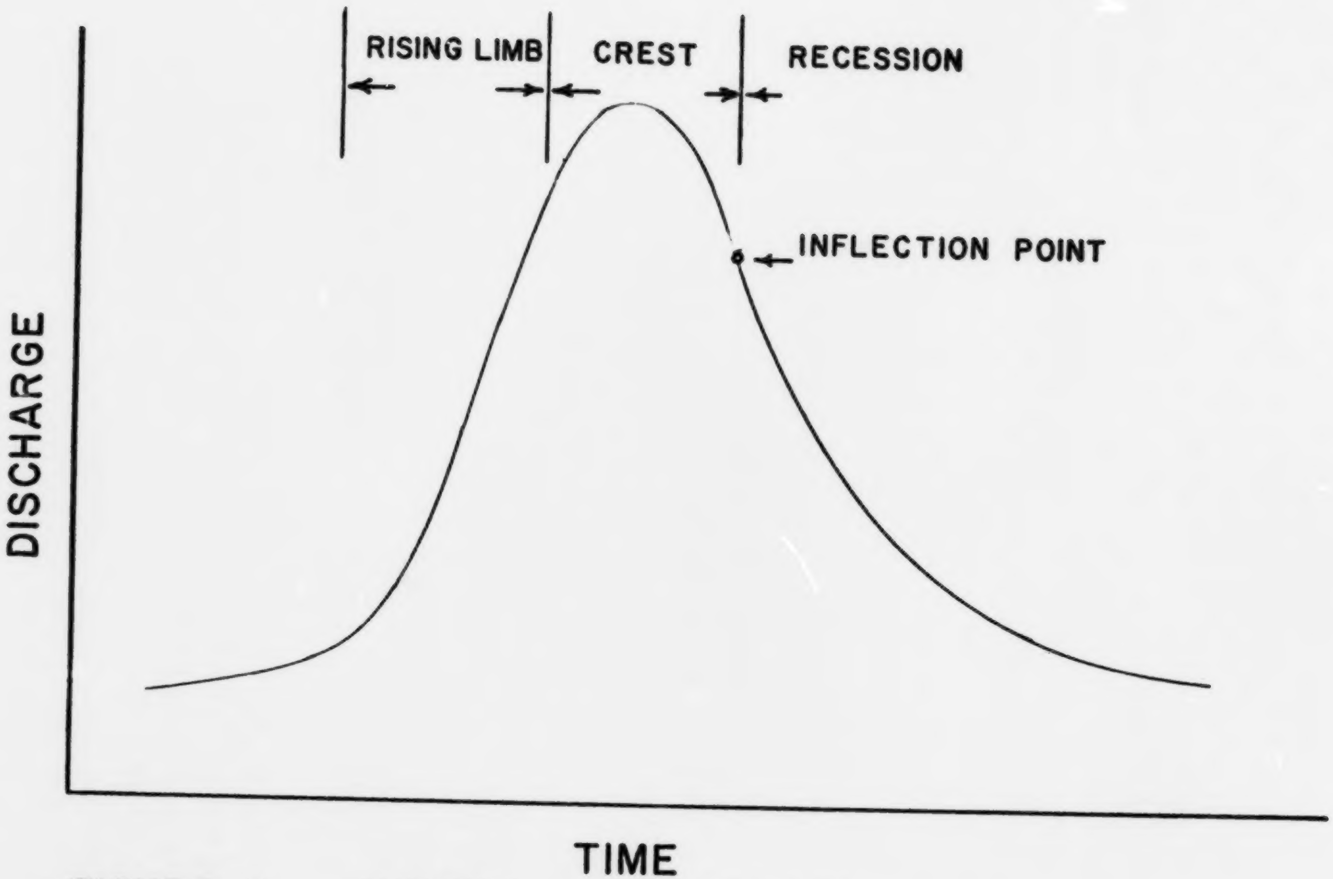

RISING LIMB
CREST
RECESSION
INFLECTION POINT
DISCHARGE
TIME

FIGURE 9. RUNOFF HYDROGRAPH

drainage system. The point of inflection on the falling side of the runoff hydrograph is commonly assumed to mark the time at which surface inflow into the main channel system has ceased.

1.5 Urban Drainage Systems

The effects of urbanization on the hydrologic response of a previously undeveloped watershed are becoming more widely known as continuing data collection programs yield more information concerning runoff quantity and quality. The effects of urbanization on peak runoff rates have been evaluated in many studies over the past several years. Carter (1961), in relating peak flow to degree of imperviousness showed that 50 to 100 percent increases may be expected. Studies by Espey *et al*. (1965) and Espey and Winslow (1968; 1969) have shown that urbanization may increase flow rates as much as four times those found for undeveloped areas. Numerous other studies (Wiitala, 1961; Van Sickle, 1962 (see Fig. 10); Crawford and Linsley, 1962; and Sawyer, 1961) have shown similarly large increases in peak flow rates. The changes in flow rates mentioned above are caused by several factors. Impervious areas such as parking lots, streets, and roofs, result in increased total runoff volumes and peak flow rates. Storm drainage and collection systems decrease the time of concentration and increase peak flow rates. The lining and rectifying of drainage ditches also may generate increased runoff rates. Such increased runoff rates due to development often result in substantial downstream flooding and property damage.

The Task Force on Effect of Urban Development on Flood Discharges, American Society of Civil Engineers has summarized these effects as follows:

A. Where the urban development encompasses the major part of a drainage area, the following effects may be felt:

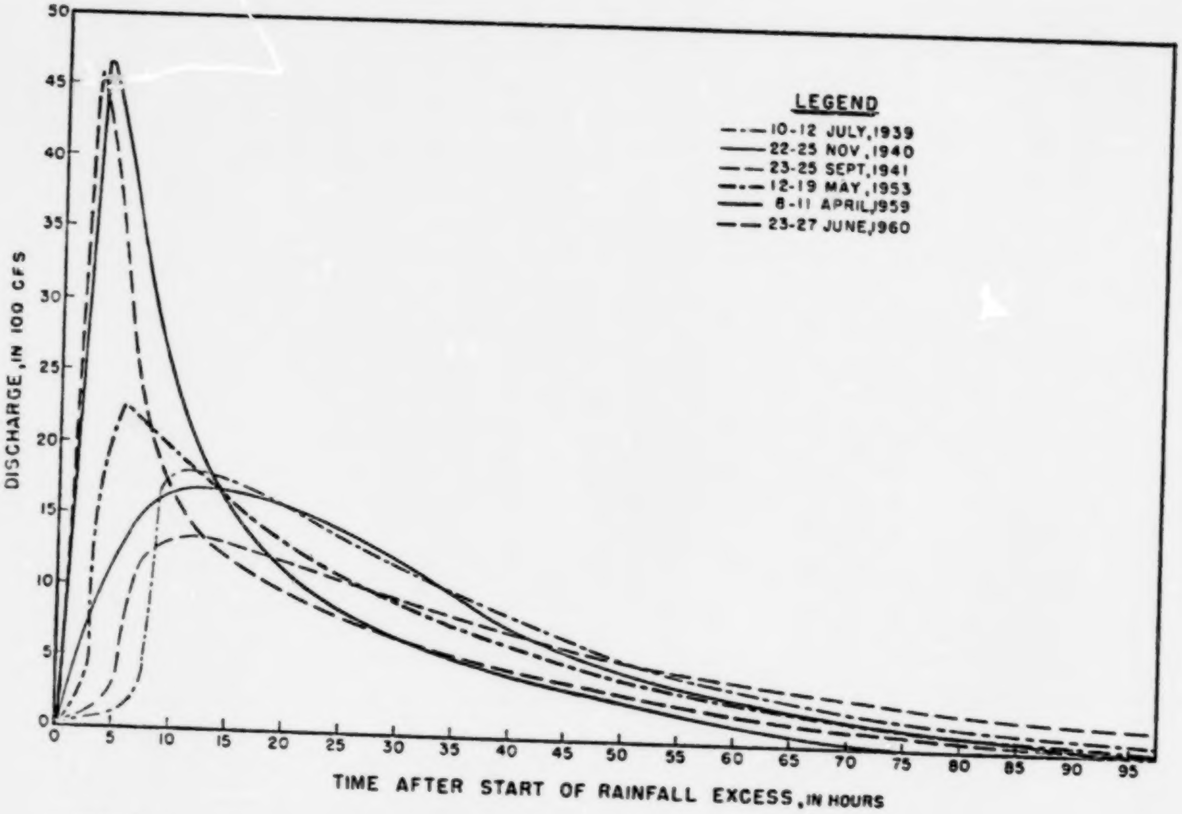


FIGURE 10. BRAYS BAYOU UNIT HYDROGRAPHS (AFTER VAN SICKLE, 1962).

- (a) Change in total volume runoff
    - (1) Annual
    - (2) Monthly
    - (3) Short-term (daily or storm)
- (b) Change in distribution of total runoff (concentration in storms; less base flow)
- (c) Change in peak rates of runoff
- (d) Change in frequency of peak rates
- (e) Change in shape of short-term hydrograph
- (f) Change in sediment content of stream
- (g) Throttling action by enclosed drainage system

B. Where urban drainage area is tributary to a river with a larger undisturbed drainage area:

- (a) Change in total runoff will depend in part on ratio of urban drainage to drainage area of river.
- (b) Change in flood peaks (magnitude and timing) will depend on ratio of areas and differences in character of areas.
    - (1) Floods in river may have two or more peaks.
    - (2) If urban area dominates, peak rates and frequencies may be changed.
- (c) Quick runoff from urban area may cause "backwater" effects and even reverse flow on main river.
- (d) Rapid rises and falls may have destructive effects on river banks.

An urban drainage system can be characterized by a high percentage of impervious cover. The drainage system will normally include man-made impervious pathways for collecting or guiding the flow of water over the surface (curbs, gutters, lined channels, paved parking areas, streets, etc.) and underground (storm, wastewater and combined sewers). The system includes all appurtenances such as catch basins, storage basins, inlets, manholes, sediment traps, ditches, wiers and outfall structures that guide, control, or otherwise modify either the quantity, rate of flow or quality of runoff from urban drainage areas. Shown in Fig. 11 is a simplified urban drainage system. The system consists of subsystems dealing with

a. surface runoff,
b. transport of flow, and
c. the receiving water.

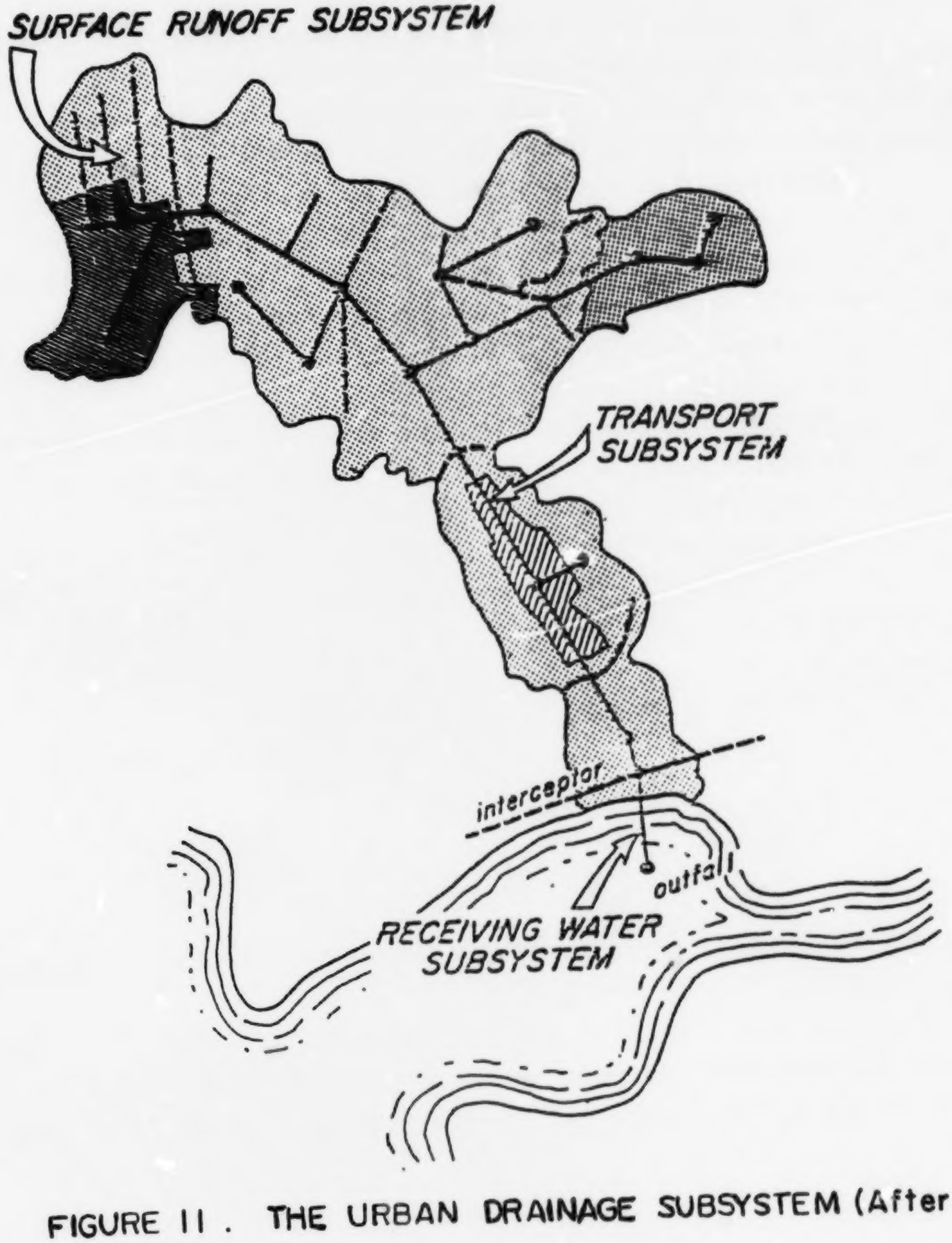

SURFACE RUNOFF SUBSYSTEM
TRANSPORT SUBSYSTEM
interceptor
outfall
RECEIVING WATER SUBSYSTEM

FIGURE 11. THE URBAN DRAINAGE SUBSYSTEM (After Orlob, 1974)

1.6 Open Channel Flow

Open channel flow consists of flow in natural streams, rivers, canals, sewers, tunnels, and pipes partially full. Problems are more difficult to solve as shape can be practically anything and roughness is often difficult to determine.

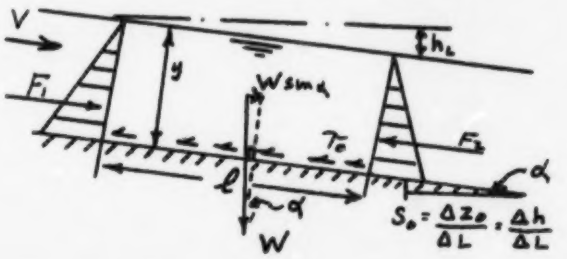
Uniform flow is established in open channels which are sufficiently long, have constant slope and cross-section, and are unaffected by backwater conditions. For any channel of given roughness, cross section, and slope, there is only one depth of flow Y_o at which the flow is uniform for a given flow rate.

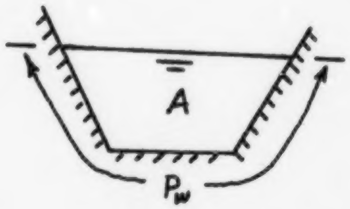
1.6.1 Slopes - Piezometric head = slope of water surface = S_w; Hydraulic Slope, = h_f/L, where h_f is the energy loss for the horizontal distance, L; Slope of the bed = S_o.

For uniform flow only

$$S = S_w = S_o \tag{23}$$

Uniform Flow - Chezy Equation


V
F_1
y
$W\sin\alpha$
h_L
τ_o
F_2
α
ℓ
α
W
$S_o = \frac{\Delta z_o}{\Delta L} = \frac{\Delta h}{\Delta L}$



Summing forces along the direction of flow in the channel schematic shown above

$$\tau_o P_w L = W\sin\alpha = ALsin\alpha \tag{24}$$

where τ_o is the shear stress, P_w the wetted perimeter and W is the weight of the water. For small angles $\sin\alpha \cong S_o$

$$\tau_o P_w L = \gamma A L S_o \tag{25}$$

$$\tau_o = \gamma \frac{A}{P} S_o \tag{26}$$

$$\tau_o = \gamma R S_o$$

where R is the hydraulic radius.
By analogy with pipe flow

$$\tau_o = \frac{f}{4} \quad V^2/2 \tag{27}$$

where f is the friction factor.
Equating the two expressions for τ_o

$$\frac{f}{8}\rho V^2 = \gamma R S_o \tag{28}$$

$$V = \sqrt{\frac{8\gamma}{\rho f}}\sqrt{RS_o} = \sqrt{\frac{8g}{f}}\sqrt{RS_o} \tag{29}$$

or $C = \sqrt{\frac{8g}{f}}$

The Chezy equation is

$$V = C\sqrt{RS_o} \tag{30}$$

1.6.2 Equations for Open Channel Flow - The most common equation is the Manning Equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{31}$$

where n is a measure of the boundary roughness. From the Chezy Equation

$$C\sqrt{RS_o} = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{32}$$

$$C = \frac{1.486}{n} \frac{R^{2/3}}{R^{1/2}} \frac{S_o{}^{1/2}}{S^{1/2}} \tag{33}$$

$$C = \frac{1.486}{n} R^{1/6} \qquad (34)$$

Other formulas for open channel flow, evaluation of the Chezy coefficient from Darcy Equation

$$C = \frac{8g}{f} \qquad (35)$$

From Manning Equation

$$C = \frac{1.486}{n} R^{1/6} \qquad (36)$$

From Hazin

$$C = \frac{157.6}{1+(m/\sqrt{R})} \qquad (37)$$

where m is a measure of absolute roughness and
$m = .11$ for smooth concrete; $m = 0.83$ for rubble masonery;
$m = 2.36$ for earth channel.
From Kutter Equation

$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \frac{n}{\sqrt{R}}\left(41.65 + \frac{0.00281}{S}\right)} \qquad (38)$$

$$C = \phi(R, n, S) \qquad (39)$$

Presented in Table II are suggested values by ASCE (1960) for Manning's (n) coefficients.

1.7 Methods of Routing

Many simplified methods of flood routing have been developed through the years, some of which have general application, while others are designed for a specific situation or problem. Most of the simplified methods are storage-routing methods and are based on a simplified form of the continuity equation for unsteady flow:

$$V \frac{\partial A}{\partial x} + A \frac{\partial V}{\partial x} + B \frac{\partial h}{\partial t} + q = 0 \tag{40}$$

where V is the average velocity
A is the cross sectional area
B is the top width of the channel
h is the water surface elevation from a given datum
x is a distance in the direction of flow
t is time
q is lateral inflow in cfs per unit length of channel

The well-known Muskingum method is a storage-routing method and is discussed in detail in paragraph 1.7.2. The unit-response method is also a storage-routing method which utilizes the principle of superposition. Other simplified methods include lag methods and mechanical and electronic machines designed for specific situations. In addition the kinematic wave theory is discussed with respect to the SWMM.

1.7.1 Basic Concepts - Change-in-storage is the basic underlying principle of most simplified methods of flow routing in natural channels. Equation (40) is the partial differential equation which contains the elements of storage routing and is sometimes called the storage-continuity equation. This equation is usually simplified to fit the type of routing to be accomplished. For instance, in reservoir routing, the concept that outflow equals inflow minus change in storage is used. Change-in-storage is usually related only to prism storage for reservoirs, whereas in open-channel routing it is usually necessary to consider wedge storage as well as prism storage.

Wedge storage and prism storage is illustrated in Fig. 12. It is assumed, of course, that a uniform, unbroken, water surface exists. This is not the case in most natural river channels but the computational methods tend to average out the differences. In Fig. 12, it can be seen that wedge storage is larger on a rising stage than on a falling stage because the water surface slope is steeper on a rising stage. In this sense, accounting for wedge storage is an indirect way of accounting for changing water-surface slope.

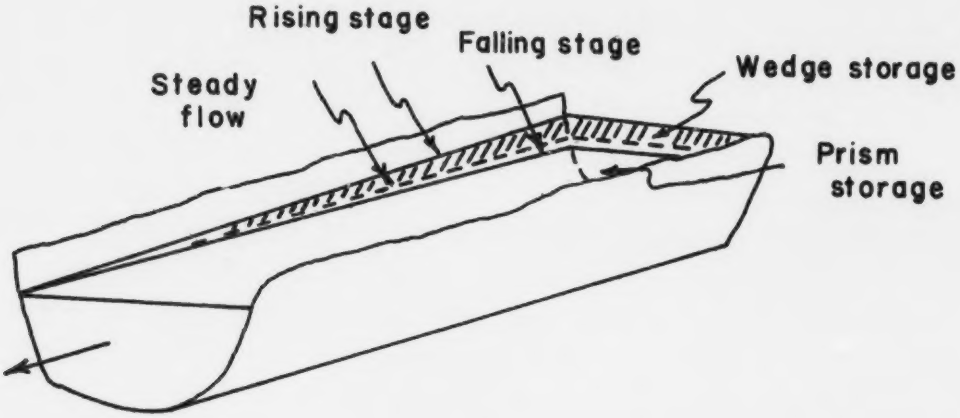


Fig. 12 Relation of Wedge Storage to Prism Storage

The relation of inflow, outflow, and storage is illustrated in Fig. 13 . Simple reservoir storage (Fig 13a) is a limiting condition of open-channel routing. Under this condition, storage is a direct function of outflow and increases as long as outflow increases. When outflow reaches a crest, the change-in-storage is zero and outflow equals inflow. After peak outflow occurs, storage decreases until outflow ceases. The other limiting condition of open-channel routing is the idealized, uniform channel as shown in Fig. 13b In this case, a flood wave moves downstream with no change in shape. Actual river channels are usually somewhere between these limiting conditions. Pools and riffles, and expansions and contractions, are in effect series of partial reservoirs which have the same attenuating effects as reservoir storage.

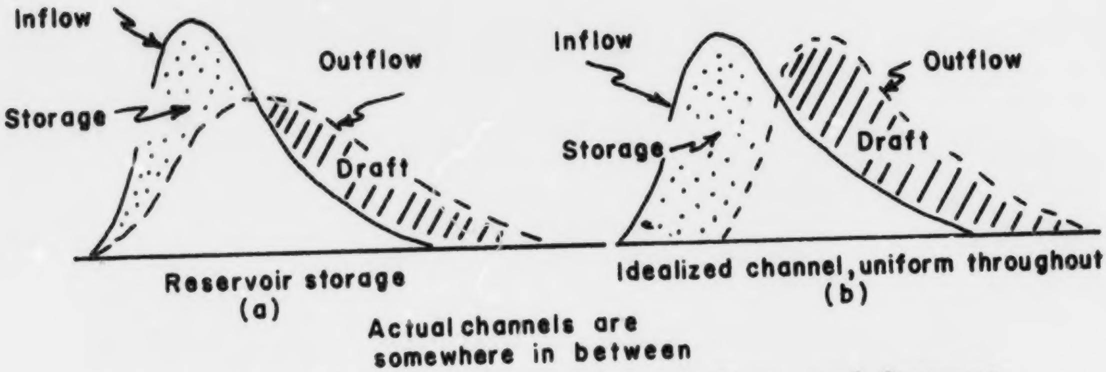

Inflow
Storage
Outflow
Draft
Reservoir storage
(a)
Inflow
Storage
Outflow
Draft
Idealized channel, uniform throughout
(b)
Actual channels are
somewhere in between

Fig. 13 Relation of Inflow, Outflow, and Storage for Flow in Open-Channels

1.7.2 Muskingum Method - The Muskingum method is based on the storage-continuity equation as in reservoir routing. The basic difference between the two methods is in the relation of storage to discharge. In reservoir routing, storage is usually considered a unique function of outflow. In the Muskingum method, storage is related to both inflow and outflow to account for wedge storage that results from changing slopes of the water surface. The relation for storage, S, is defined as follows:

$$S = K \left[xI + (1-x)\ 0 \right] \tag{41}$$

where K the slope of the discharge-storage relation, has the dimension of time, and represents the time required for the center of mass of the flood wave to traverse the reach, x is a dimensionless constant which weights inflow and outflow, I is the inflow rate, and O is the outflow rate.

Combining equation (41) with the storage-continuity equation results in the following routing equation:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \tag{42}$$

where

$$C_o = - \frac{Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t} \tag{43}$$

$$C_1 = \frac{Kx + 0.5\ \Delta t}{K - Kx + 0.5\ \Delta t}\ , \tag{44}$$

and

$$C_2 = \frac{K - Kx - 0.5\ \Delta t}{K - Kx + 0.5\ \Delta t}\ . \tag{45}$$

Application of equation (42) to a routing problem is a simple process because C_o, C_1, and C_2 are constants. In most cases, K is considered constant for the full range of discharges to be routed, however it can be varied if significant differences are defined. The constant x is always used as a non-variable. The range of x is 0 to 0.5, with x = 0 being reservoir conditions and x = 0.5 being idealized uniform channel conditions resulting in pure translation (no change in shape). Substitution of x = 0 in equation (43), (44), and (45) will result in the identical equation defined for reservoir routing. Most natural channels have values of x in the range of 0.2 to 0.3. Techniques of estimating K and x from actual records are given by Chow (1964).

A critical factor in the use of the Muskingum method is the selection of Δt, the computation interval. This time interval must be greater than 2Kx to avoid negative values of C_o, but for some reaches this criteria will result in a Δt too long to adequately define the hydrograph. In these cases, the reach must be subdivided into shorter subreaches and a stepwise routing performed. The outflow from the first subreach becomes the inflow to the next subreach, and so on until the downstream site is reached. The value of K for each subreach is made proportional to the length of the subreach.

1.7.3 Kinematic Waves and Their Limited Application in the SWMM - A kinematic wave is one in which Q, the flow rate, in a given channel is a function of depth, y, alone as is assumed in uniform flow computations.

Using the Chézy equation to elaborate:

$$Q = CA\sqrt{RS_f} \qquad S_f = \text{friction slope} \tag{46}$$

rewriting and expanding S_f into components

$$Q = CA\sqrt{R\left(S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t}\right)} \tag{47}$$

where S_o = bottom slope

$$\frac{\partial y}{\partial x} = \text{water surface slope} \tag{48}$$

$$\frac{v}{g}\frac{\partial v}{\partial x} = \text{velocity head slope} \tag{49}$$

$$\frac{1}{g}\frac{\partial v}{\partial t} = \text{acceleration term (unsteady flow)} \tag{50}$$

If the last three terms are small compared to S_o the assumption that $S_o = S_f$ can be made. Henderson (1966) offers typical values to be $26\ \frac{ft}{mi}$, $\frac{1}{2}\ \frac{ft}{mi}$, $\frac{1}{8}\ \frac{ft}{mi}$, and $\frac{1}{20}\ \frac{ft}{mi}$, respectively. Hence it seems that the $S_o = S_f$ statement is generally applicable.

Although it has been shown that Q can reasonably be assumed to be a function of y alone, it remains to be seen that a flood wave can be described in this manner. Let the continuity equation be written in the form:

$$\frac{\partial Q}{\partial x} + B\,\frac{\partial y}{\partial t} = 0 \qquad \text{where } B = \text{width} \tag{51}$$

introducing the total derivative

$$\frac{dQ}{dy}\;\frac{\partial y}{\partial x} + B\,\frac{\partial y}{\partial t} = 0 \tag{52}$$

eleminating B and dQ

$$\frac{dy}{dt} = \frac{dx}{dt}\;\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \tag{53}$$

rearranging

$$\frac{dx}{dt} = C = \frac{1}{B}\,\frac{dQ}{dy} \tag{54}$$

where C = wave celerity.

From this it is seen that an observer moving with velocity C will see Q as a function of y alone.

Due to the simplistic nature of the equations, kinematic theory can be easily implemented in computer formulations. Accordingly, an attempt was made by the initial developers of the SWMM to implement such a scheme into the TRANSPORT routing technique.

This was attempted by developing dimensionless parameters α and ψ which are defined respectively as $\frac{A}{A_f}$ and $\frac{Q}{Q_f}$ where the subscript f denotes "full". Now if Mannings' equation is introduced using the assumption $S_o = S_f$

$$Q = \frac{1.49}{n}\,AR^{2/3}\,S_o^{\;1/2} \tag{55}$$

If the above equation is normalized by the full-flow values of Q, A, and R, there results

$$\psi \equiv \frac{Q}{Q_f} = \frac{A\, R^{2/3}}{A_f\, R_f^{2/3}} = f(\alpha) \qquad (56)$$

The normalized flow rate will be a function only of the normalized area since the hydraulic radius is a known function of the flow area for a given geometry.

According to the Final Report, this method produced reasonable hydrographs in conduits with steep slope (> .001) but not for mild slopes.

For this reason, it was found necessary to deviate from the $S_f = S_o$ approximation by adding two dynamic terms $\frac{\partial y}{\partial x}$ and $\frac{v}{g}\frac{\partial v}{\partial x}$. Obviously, the method no longer produces a strictly kinematic result on mild slopes. The method of implementation in the computer program allows for the invert slope S_o to be used alone if S_o is of sufficient magnitude.

2.0 URBAN HYDROLOGY - STATE OF THE ART

In general, urban hydrology literature consists of a maze of empirical, statistical and theoretical methods which in many cases are more art than science. Many urban design techniques consist of a combination of these methods. For the purposes of this review, the urban hydrology literature has been segregated into the following categories:

1) Empirical/Statistical Formulas,
2) Flood Frequency Analysis, and
3) Simulation Methods

2.1 Empirical/Statistical Formulas

Empirical formulas are, in general, based on some form of hydrologic hypothesis with resulting relationships developed by empirical derivation of equations and/or coefficients. Empirical formulas are by far the oldest and still the most widely used method of predicting peak flows from urban watersheds. Of the many empirical formulas (Hawksley, 1857; Adams, 1880; McMath, 1887; Parmely, 1905; Gregory, 1907; and historical review by Chow, 1962) that have evolved since the early 1800's, the so-called "rational" method has continued to be the most widely used method in the United States for estimating peak flows from small urban watersheds (Ardis, Dueker, and Lent, 1969). The "rational" method was first proposed in Great Britain by Lloyd-Davies in 1905; however, this same method appears to have been applied in Ireland as early as 1850 by Mulvaney and in the United States by Kuichling in 1889. Because of the wide use of the "rational" method for storm sewer design, the literature abounds with papers on this method. Various hydrologists, such as Horner and Jens (1942), Potter (1950), and Watkins (1962), have pointed out the limitations of the "rational" method. Meek (1928) found the runoff coefficient used in the "rational" method is not a constant, but varies with the characteristics of the storm and previous surface moisture conditions of the watershed. MacDonald and Mehn (1963) investigated the rational runoff coefficients for residential areas in New Orleans. Summarized in Tables III and IV are rational runoff coefficients reported by various investigators and the ASCE. More recently, Schaake, Geyer, and Knapp (1964), as part of the Storm Drainage Research Project at Johns Hopkins University, evaluated the application of the "rational" method to 20 urban watersheds. Results from study were, in general, inconclusive.

The basic theory of the unit hydrograph appears to have been suggested first by Folse (1929). The Boston Society of Civil Engineers (1930) stated, "the base of the flood hydrograph appears to be approximately constant for different floods, and peak flow tends to vary directly with the total volume of runoff". Three years later, in 1932, Sherman introduced the basic concept of the unit hydrograph. The unit hydrograph has become widely accepted as one of the outstanding contributions to the science of hydrology. The general concept of the unit hydrograph can be summarized as follows (Morgan and Johnson, 1962):

1) For a given drainage area, the time base of surface-runoff hydrographs resulting from similar storms of equal duration are the same regardless of the intensity of rainfall.

2) For a given drainage area, the ordinates of the surface-runoff hydrographs from similar storms of equal duration are proportional to the volume of surface runoff.

3) For a given drainage area, the time distribution of surface runoff from a particular storm period is independent of that produced by any other storm period.

Because the unit hydrograph is such a valuable hydrologic tool, many investigations have been undertaken to develop synthetic

TABLE III

COMPARISON OF RATIONAL FORMULA RUNOFF COEFFICIENTS
(after MacDonald and Mehn, 1963)

Reference		**Type of Area**			
		Commercial	***Residential***	***Farm or Cleared Land***	***Wooded Area***
H. E. Babbitt	(a)	0.70-0.90	0.45-0.70	0.05-0.25	0.01-0.20
E. E. Seelye	(b)	0.60-0.75	0.35-0.55	0.10-0.35	0.10-0.25
L. C. Urquhart	(c)	0.60-0.95	0.25-0.40	... ...	... ...
Fruhling	(d)	0.70-0.90	0.20-0.50	0.05-0.25	0.01-0.20
ASCE	(e)	See Note (f)	0.25-0.40	0.10-0.25	0.10-0.30

(a) Babbitt, 1956
(b) Seelye, 1960
(c) Urquhart, 1959
(d) Metcalf and Eddy
(e) ASCE, 1960
(f) 0.70-0.95 for Downtown areas;
0.50-0.70 for Suburban areas.

TABLE IV

RUNOFF COEFFICIENT VALUES RECOMMENDED BY THE ASCE
(after MacDonald and Mehn, 1963)

Character of Surface	**Run-off Coefficients**	
Streets:		
Asphaltic	(0.83)	0.70-0.95
Concrete	(0.88)	0.80-0.95
Drives and Walks	(0.80)	0.75-0.85
Roofs	(0.85)	0.75-0.95
Lawns:		
Sandy Soil, Flat 2%	(0.08)	0.05-0.10
Heavy Soil, Flat 2%	(0.15)	0.13-0.17
Parks	(0.18)	0.10-0.25

unit graphs from relationships based on watershed features. The watershed features commonly used in development of these relationships are watershed area, shape, topography, channel slope, stream density, and channel storage. Of the various synthetic unit hydrograph methods that have been developed, probably the work by Snyder (1938), USSCS (1957), Commons (1942), and Mitchell (1948) are best known and most widely used. In particular, the Snyder method has been an integral part of the hydrologic design procedures of the USCE.

Chow (1962) developed a procedure to generate the runoff hydrograph based on the unit hydrograph. Using data for Illinois watersheds, he developed a procedure to define the unit hydrograph which, in combination with a rainfall-runoff relationship based on the SCS technique, yields the total runoff hydrograph.

Application of the unit hydrograph concept to urban sewer systems was first reported by Horner and Flynt (1936). Predicted hydrographs were in good agreement with observed data; however, the data base was small, consisting of only three small (less than 5 acres) urban watersheds. Tippetts, Abbett, McCarthy and Stratton, Consulting Engineers, successfully applied the unit hydrograph to runoff calculations for the city of Philadelphia in 1947. Van Sickle (1962) used the unit hydrograph as an integral part of the development of urban design criteria for the city of Houston. Eagleson (1962), using USCE hydrologic data for five small (0.22 to 7.52 sq.mi.) urban watersheds located in Louisville, Kentucky, investigated the application of the unit hydrograph concept to these watersheds. Espey, Morgan, and Masch (1965), while studying the various effects of urbanization on the hydrologic characteristics of a small urban watershed located in Austin, Texas, developed a set of synthetic unit hydrograph equations (Table V) based on data from 24 urban watersheds located throughout the United States. Based on additional hydrologic data from 11 urban watersheds located in Houston, Texas,

TABLE V

URBAN EQUATIONS
(Espey, Morgan, and Masch, 1965)

$$T_{RU} = 20.8\,\Phi\,L^{0.29}\,S^{-0.11}\,I^{-0.61} \quad ,$$

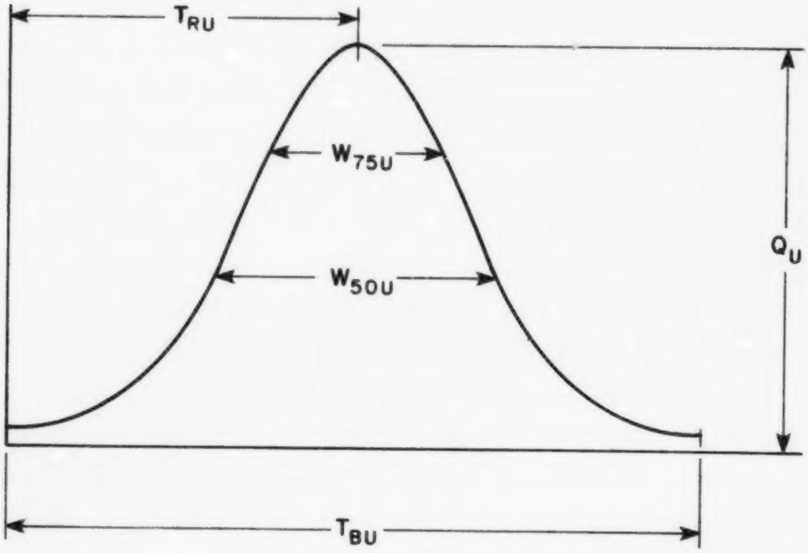
$$Q_U = 1.93 \times 10^4\,A^{0.91}\,T_{RU}^{-0.94} \quad ,$$

$$T_{BU} = 4.44 \times 10^5\,A^{1.17}\,Q^{-1.19} \quad ,$$

$$W_{50U} = 4.14 \times 10^4\,A^{1.03}\,Q^{-1.04} \quad ,$$

and

$$W_{75U} = 1.34 \times 10^4\,A^{0.92}\,Q^{-0.94} \quad .$$



Espey and Winslow (1968) tested and modified the unit hydrograph equations developed by Espey et al. (1965). The modified equations are shown in Table VI.

Watkins (1962) developed a method (RRL) in England describing the runoff from urban sewer systems. The RRL method, in general, is based on the same principles as the unit hydrograph method. The RRL method assumes that pervious area does not produce runoff, and that impervious area yields 100% runoff. A time-area diagram of directly connected impervious area of the basin is constructed and the resulting runoff is routed through a linear reservoir. Terstriep and Stall (1969) report testing on three urban watersheds located in Baltimore, Maryland, Chicago, and Champaign, Illinois, using the RRL method with good results. For 286 storms measured on 12 watersheds located in England, Watkins (1962) also evaluated the application of the following urban design methods:

1) Rational (Lloyd-Davies) formula,
2) Tangent method,
3) Modified Tangent method, and
4) Unit Hydrograph method;

Watkins' evaluation of these methods can be summarized as follows:

1) "The rational method is unsatisfactory for all but the smallest areas. For sewer systems containing pipes not larger than 24 inches in diameter, the range of available pipe sizes is such that the errors introduced by the use of this method are tolerable. It was found that when the "rational" formula gives an accurate estimate of the peak rate of runoff, it does so because there is, for the particular area concerned, a rough balance between the error introduced by neglecting the shape of the area/time diagram and the error introduced by neglecting the reservoir retention in the sewer system."

2) "The tangent and modified tangent methods were found to be normally no more accurate than the "rational" formula, and it was also found that in some circumstances the use of these methods could increase the errors."

TABLE VI

URBAN EQUATIONS
(Espey and Winslow, 1968)

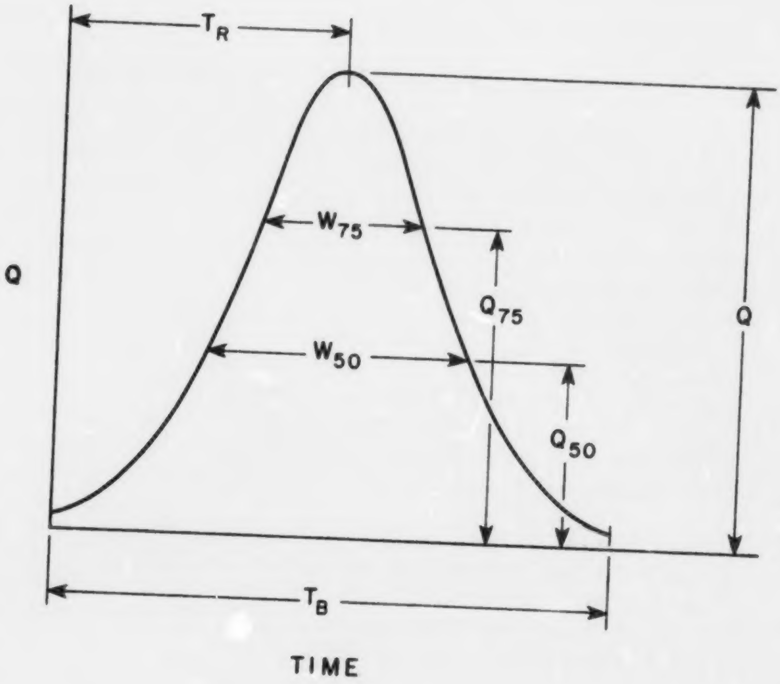
$$T_R = 16.4\ \phi\ L^{0.315}\ S^{-0.0488} I^{-0.490}$$

$$Q = 3.54 \times 10^4\ T_R^{-1.10}\ A^{1.00}$$

$$T_B = 3.67 \times 10^5\ A^{1.14}\ Q^{-1.15}$$

$$W_{50} = 4.14 \times 10^4\ A^{1.03}\ Q^{-1.04}$$

$$W_{75} = 1.34 \times 10^4\ A^{0.92}\ Q^{-0.94}$$


T_R
Q
W_75
Q_75
Q
W_50
Q_50
T_B
TIME

3) The unit hydrograph method, in the form usually employed by hydrologists for calculating streamflow in natural catchments, was shown to be unsuitable for designing urban sewer systems owing to the difficulty of determining the shapes of the unit hydrographs."

Other empirical equations have been developed for overland flow by Izzard (1946), Kerky (1959), Brater and Woo (1962), Morgali (1963), Henderson and Wooding (1964), Grace and Eagleson (1966), and Woolhiser and Liggett (1967). These methods all permit better definitions of the time to equilibrium and the peak flow.

At Johns Hopkins University, Viessman and Geyer (1962) developed equations which relate the peak flow and time of rise to the rainfall rate, drainage area, and other physiographic parameters. Equations were also developed to describe the concentration and recession curves for the storm hydrograph. The equations were developed from data for very small and mostly impervious areas.

2.2 Flood Frequency Analysis

Carter (1961) developed an empirical equation relating the mean annual flood to the lag time, drainage area and percent of impervious cover to determine the effect of urbanization on the mean annual flood in the vicinity of Washington, D. C. This equation is:

$$\overline{Q} = 223\ K\ A^{0.85} T^{-0.45} \qquad (57)$$

where $\overline{Q}$ is the mean annual flood in cubic feet per second and is equivalent to the flood having a recurrence interval of 2.33 years, A is the drainage area in square miles, T is the lag time expressed in hours, and K is an adjustment factor based upon the degree of imperviousness of the area. The factor K is expressed as:

$$K = \frac{0.30 + 0.0045\ I}{0.30} \qquad (58)$$

where I is the percent of impervious cover.

Martens (1961) further investigated the application of Carter's equations to some small urban watersheds located in Charlotte, North Carolina. Anderson (1968) also applied Carter's equations to selected urban watersheds located on the east coast. For a small urban watershed (4.1 sq.mi.) located in Austin, Texas, Espey _et al_. (1965) found good agreement between Carter's predicted mean annual flood and measured data.

Putnam (1972) developed relationships between peak discharges for various recurrence intervals and physiographic and urban parameters for approximately 200 watersheds in the North Carolina Piedmont Province of which 42 are urban. The flood frequency equations relate the flood peak discharge for various recurrence intervals with lag time and the drainage area. The flood frequency equations for the 50 and 100-year flood were derived by extrapolating the regression coefficients, based on the regression coefficients for the 2, 5, 10, and 25-year flood frequency equations. The equations are as follows:

$$P_2 = 221\ A^{0.87}\ T^{-0.60} \qquad (59)$$

$$P_5 = 405\ A^{0.80}\ T^{-0.52} \qquad (60)$$

$$P_{10} = 560\ A^{0.76}\ T^{-0.48} \qquad (61)$$

$$P_{25} = 790\ A^{0.71}\ T^{-0.42} \qquad (62)$$

$$P_{50} = 990\ A^{0.67}\ T^{-0.37} \qquad (63)$$

$$P_{100} = 1200\ A^{0.63}\ T^{-0.33} \ldots\ldots\ldots\ldots\ldots\ldots (64)$$

where,

P_i is the peak discharge in cubic feet per second for the flood having the recurrence interval indicated by the subscript,

A is the drainage area in square miles, and

T is the lag time in hours.

The lag time is obtained from the following equation:

$$T = 0.49\ (L/\sqrt{S})^{0.50}\ I^{-0.57} \ldots\ldots\ldots\ldots\ldots\ldots (65)$$

where,

T is the lag time in hours,

L is the length of the main water course in miles,

S is the stream bed slope of the main water course in feet per mile, and

I is the ratio of the area of impervious cover to the total drainage area.

The flood frequency equations developed by Putnam (1972) are based on limited urban data. Because of the limited historical data used in this analysis, extrapolation to the 50 and 100-year flood should be exercised cautiously.

Benson (1964) studied the relationship between flood peaks and physiographic and hydrographic factors for primarily rural basin in the Western Gulf of Mexico Basin. The study area comprised most of Texas and New Mexico, and small parts of Louisiana and Colorado. Multiple regression analysis was used to develop relationships between peak discharge for various recurrence intervals and topographic hydrologic units; first, the area where the annual peaks were caused primarily by local thunderstorms or widespread tropical storms, and second, the relatively small area where the annual flood peaks were caused almost entirely by snowmelt. Peak discharges within the rainstorm-flood area were found to be significantly related to seven factors: drainage area, rainfall intensity for a given duration and frequency, main-channel slope, basin length, surface area of lakes and ponds, the ratio of runoff to rainfall during the months of annual peak discharge, and the annual number of thunderstorm days. The last two factors, although statistically significant, play only a small part in the variability of flood peaks. It may be expected that some index of rainfall that involved a short duration and an element of frequency would be highly correlated with the peak discharge of a given frequency.

Flood frequency equations were developed by Espey and Winslow (1972) which predict the peak flow for urban basins for selected recurrence intervals. The equations were based on Log-Pearson Type III flood frequency analysis from which the 2.33, 5, 10, 20, and 50-year recurrence interval peak flows for each watershed were determined. Data for sixty watersheds located in Texas, Virginia, Maryland, Delaware, Michigan, Illinois, and Mississippi were used in the multiple regression analysis. The independent variables used in the analysis were as follows: 1) drainage area (A) sq. mi., 2) slope of the main channel (S), ft./ft., 3) impervious cover (I), percent, 4) channel urbanization factor (I), dimensionless 5) rainfall (R), inches for 6-hour duration for the approximate recurrence interval (U. S. Weather Bureau, 1961). The resulting flood frequency equations obtained by Espey and Winslow (1972) are shown in Table VII. The correlation coefficients shown are based upon the logarithms of the data, while the average percent errors are based on the actual flow data rather than the logs. It should be noted that the channel urbanization factor becomes more significant and the percent impervious cover becomes slightly less significant as the recurrence interval increases.

TABLE VII

DERIVED FLOOD FREQUENCY EQUATIONS
Urban Watersheds

	Correlation Coeff (Logs)	Average % Error
$Q_{2.33} = 169A^{0.77}I^{0.29}S^{0.42}R_{2.33}^{1.80}\Phi^{-1.17}$	0.97	30
$Q_5 = 172A^{0.80}I^{0.27}S^{0.43}R_5^{1.73}\Phi^{-1.21}$	0.97	31
$Q_{10} = 178A^{0.82}I^{0.26}S^{0.44}R_{10}^{1.71}\Phi^{-1.32}$	0.96	31
$Q_{20} = 243A^{0.84}I^{0.24}S^{0.48}R_{20}^{1.62}\Phi^{-1.38}$	0.96	32
$Q_{50} = 297A^{0.85}I^{0.22}S^{0.50}R_{50}^{1.57}\Phi^{-1.61}$	0.96	34
$Q_5 = 1.13Q_{2.33}^{1.03}$	0.99	8
$Q_{10} = 1.24Q_{2.33}^{1.05}$	0.99	16
$Q_{20} = 1.34Q_{2.33}^{1.06}$	0.99	22
$Q_{50} = 1.47Q_{2.33}^{1.08}$	0.97	28

2.3 Simulation Analysis

Simulation methods are generally characterized by their attempt to describe in a somewhat analytical fashion the various physical processes involved in the generation of runoff. The methods usually consist of a series of "loss" functions, which describe the amount of infiltration and the amount of interception storage, and routing procedures which describe overland and channel flow. Presented in the following paragraphs is a general summary of most of the current simulation methods.

One of the earliest simulation methods was the Stanford Watershed Model (Linsley and Crawford, 1960). The model, developed for digital computer use, simulates both surface and groundwater phenomenon, overland flow, channel routing, and snowmelt. The model employs 21 parameters for rainfall runoff and 9 additional parameters for snowmelt. Several of the input parameters are determined by trial for each watershed to be simulated. Many modifications of the model have been made (James, 1970; Liou, 1970; Claborn and Moore, 1970; and Anderson, 1968) and will not be discussed here.

The Soil Conservation Service (1971) has developed a runoff model which is primarily applicable to rural watersheds. The model is essentially a linear reservoir routing routine. The runoff processes are defined by a series of 9 hypothetical reservoirs through which the flow is routed. The reservoirs represent the various processes involved, such as infiltration, groundwater recession, and storage.

An electric analog computer model has been developed at Utah State University (Narayana *et al.*, 1969; Evelyn *et al.*, 1970). The model is operated for a series of sub-watersheds for which rainfall excess is determined by calculated infiltration, interception and depression storage. Overland and channel flow are determined by a linear routing scheme.

A digital computer runoff model has been developed by the University of Cincinnati (1970). The model consists of a series of computational schemes to determine rainfall excess, based on extensive literature review, a simplified version of the two-dimensional overland flow equations, and Mannings equations for pipe and channel flow.

One of the newest simulation models is the Storm Water Management Model developed by Metcalf and Eddy et al. (1971) for EPA. The model is designed to be a comprehensive tool to model both the quantity and quality of runoff, the impact of the runoff on the receiving water quality, and the effect of treating the stormwater. The runoff part of the model utilizes an infiltration loss function, Mannings' equation, and the continuity equation to determine the flow rate both spatially and temporally. The model is currently being used in several EPA projects, as outlined by Fields and Struzeski (1972), and is also being used in various regions of the country for stormwater design purposes.

The Chicago Hydrograph Method developed by Tholin and Keifer (1959) is essentially the overland flow routing method developed by Izzard (1946) and a storage routing method for storm sewers. The method considers infiltration losses and depression storage in determining rainfall excess.

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QUALITY ASPECTS OF URBAN STORMWATER RUNOFF

BY

Larry A. Roesner*

BACKGROUND

It has only been about ten years since sanitary engineers began to realize the significance of urban runoff as a source of pollution in receiving waters. Historically [1], the earliest sewers were built for the collection and disposal of storm runoff. For convenience, these sewers discharged to the nearest watercourse. In later years, domestic and industrial wastewaters were discharged into these sewers, thereby converting them to the combined sewers. As the significance of the pollutional effects of discharging raw sewage to the watercourses became recognized, the major cities embarked upon programs of interceptor sewers to divert some multiple (generally 1.5-5) of the average dry weather flow to a central location for treatment prior to disposal.

Even with interceptors however, stormwater overflows from these combined systems were still observed to carry significant pollution loads. This fact caused water pollution control agencies to begin thinking that separation of storm runoff and sanitary sewage was the answer to the pollution problem and for several years (about the mid-1960's) there were many studies on alternative methods of separating sanitary sewage and storm water in existing combined systems. Sewer separation was found to be very expensive, however [2], and so, while a few cities undertook separation programs, most cities began to look for alternative methods of dealing with the problem. Perhaps the most significant result to come out of this push for separation is that combined sewer systems are no longer designed for new developments; separate systems are installed.

While the push for sewer separation was on, the federal government was sponsoring research on the quality characteristics of urban runoff *per se*. Tables 1 and 2 (taken from Reference 1) were compiled from these studies and show the comparison between the characteristics of combined sewer overflow and urban stormwater. It is apparent that the BOD and solids loads in urban runoff are significantly higher than those found in combined overflows which represent a mixture of sanitary sewage and stormwater.

As a result of these findings, the emphasis on stormwater pollution control is now being placed on controlling the pollutant load of all stormwater discharges whether they emanate from a combined system or from a separate storm sewer system.

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*Principal Engineer, Water Resources Engineers, Walnut Creek, California.

TABLE 1
Characteristics of Combined Sewer Overflows*

Characteristic	Range of Values
BOD_5 (mg/l)	30–600
TSS (mg/l)	20–1,700
TS (mg/l)	150–2,300
Volatile TS (mg/l)	15–820
pH	4.9–8.7
Settleable solids (ml/l)	2–1,550
Organic N (mg/l)	1.5–33.1
NH_3N (mg/l)	0.1–12.5
Soluble PO_4 (mg/l)	0.1–6.2
Total coliforms (no./100 ml)	$20{,}000–90 \times 10^6$
Fecal coliforms (no./100 ml)	$20{,}000–17 \times 10^6$
Fecal streptococci (no./100 ml)	$20{,}000–2 \times 10^6$

* Selected data.

TABLE 2
Characteristics of Urban Stormwater*

Characteristic	Range of Values
BOD_5 (mg/l)	1–>700
COD (mg/l)	5–3,100
TSS (mg/l)	2–11,300
TS (mg/l)	450–14,600
Volatile TS (mg/l)	12–1,600
Settleable solids (ml/l)	0.5–5,400
Organic N (mg/l)	0.1–16
NH_3N (mg/l)	0.1–2.5
Soluble PO_4 (mg/l)	0.1–10
Total PO_4 (mg/l)	0.1–125
Chlorides (mg/l)	2–25,000†
Oils (mg/l)	0–110
Phenols (mg/l)	0–0.2
Lead (mg/l)	0–1.9
Total coliforms (no./100 ml)	$200–146 \times 10^6$
Fecal coliforms (no./100 ml)	$55–112 \times 10^6$
Fecal streptococci (no./100 ml)	$200–1.2 \times 10^6$

* Selected data.
† With highway deicing.

SOURCES OF POLLUTANTS

Basically, pollutant loads are introduced into urban runoff from three sources:

1. The land surface itself, primarily impervious surfaces;
2. Catch basins; and
3. The sewers in combined systems.

Of these three sources, the land is the most important.

Catch basins can be a source of first-flush or shock pollution. An American Public Works Association (APWA) study [3] in Chicago found that:

> ". . . the liquid remaining in a basin between runoff events tends to become septic and that the solids trapped in the basin take on the general characteristics of septic or anaerobic sludge. The liquid in catch basins is displaced by fresh runoff water in the ratio of one-half the volume for every equal volume of added liquid. During even minor rainfall or thaw this displacement factor can release the major amount of the retained liquid and some solids. The catch basin liquid was found to have a BOD content of 60 ppm in a residential area. For even minor storms, the BOD of the catch basin liquid would be seven-and-one-half (7-1/2) times that of the runoff which had been in contact with street litter. Improved design of catch basins, and better operational and maintenance practices, could reduce this first-flush pollutional effect."

In combined sewer systems, wastewater is incorporated into the storm runoff. In addition, the storm runoff, as it passes through large sewers, scours sediment deposited by wastewater flows during preceding dry-weather periods. Figures 1 and 2 illustrate the effects of wastewater sewage and of catch basins and storm sewer scour on the quality of stormwater overflows [4].

As stated above, the most important contributor of pollutants to urban runoff is the land surface itself, primarily the streets and gutters and other impervious areas directly connected to streets or storm sewers. Pollutants accumulate on these surfaces in a variety of ways. There is, for example: debris dropped or scattered by individuals; sidewalk sweepings; debris and pollutants deposited on or washed into streets from yards and other indigenous open areas; wastes and dirt from building and demolition; fecal dropping from dogs, birds and other animals; remnants of household refuse dropped during collection or scattered by animals or wind; dirt, oil, tire and exhaust residue contributed by automobiles; and fallout of air pollution particles. The list could go on and on. Irrespective of the way in which pollutants accumulate on the urban watershed, they are generally associated with one of the following forms of street litter:

1. Rags,
2. Paper,
3. Dust and dirt,
4. Vegetation, or
5. Inorganics.

Table 3, which gives estimated street litter components for a residential area in Chicago, provides a rough measure of the relative importance of these components.

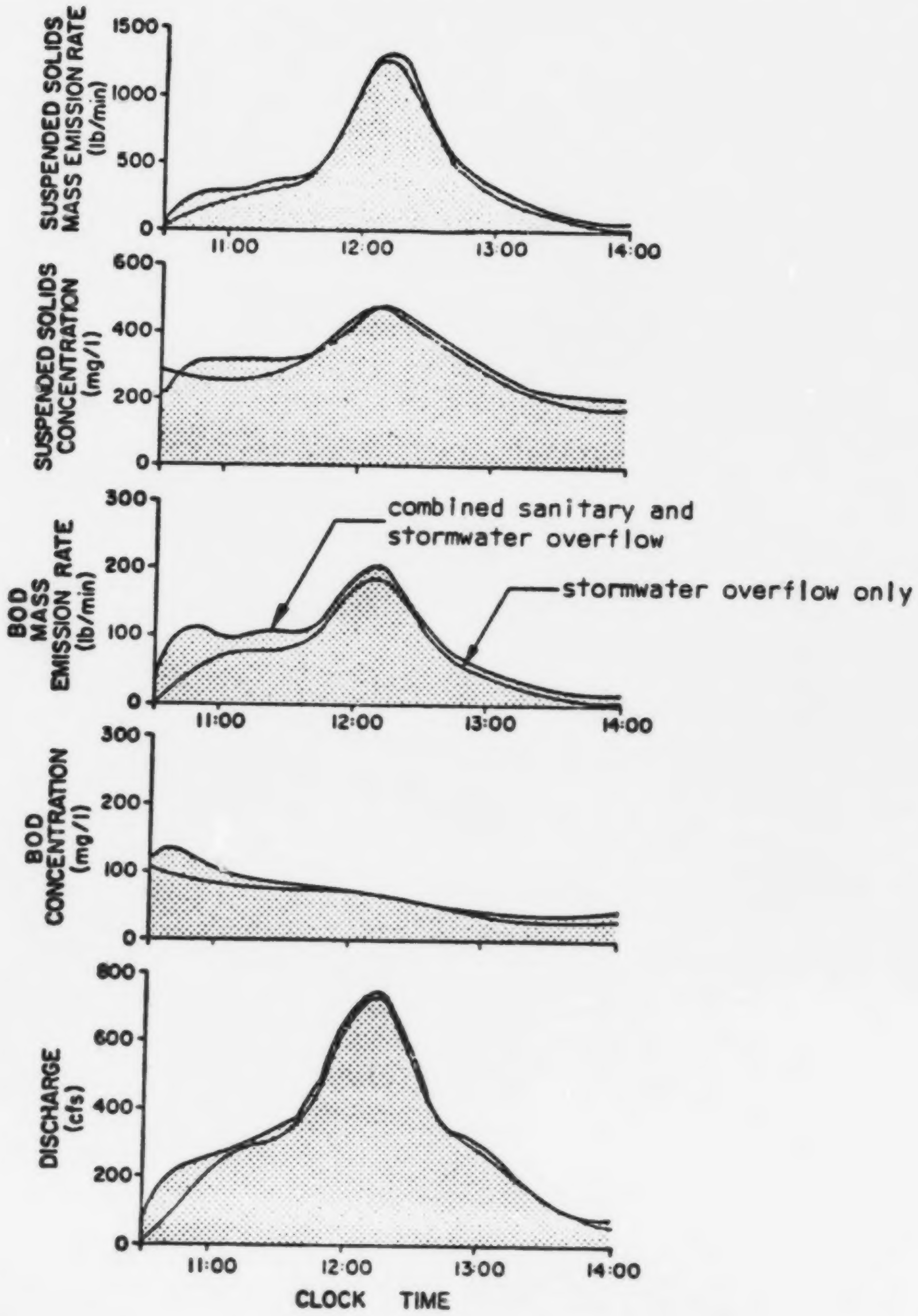

FIGURE 1
Comparison of Stormwater and Combined Overflows
(Selby Street, San Francisco)

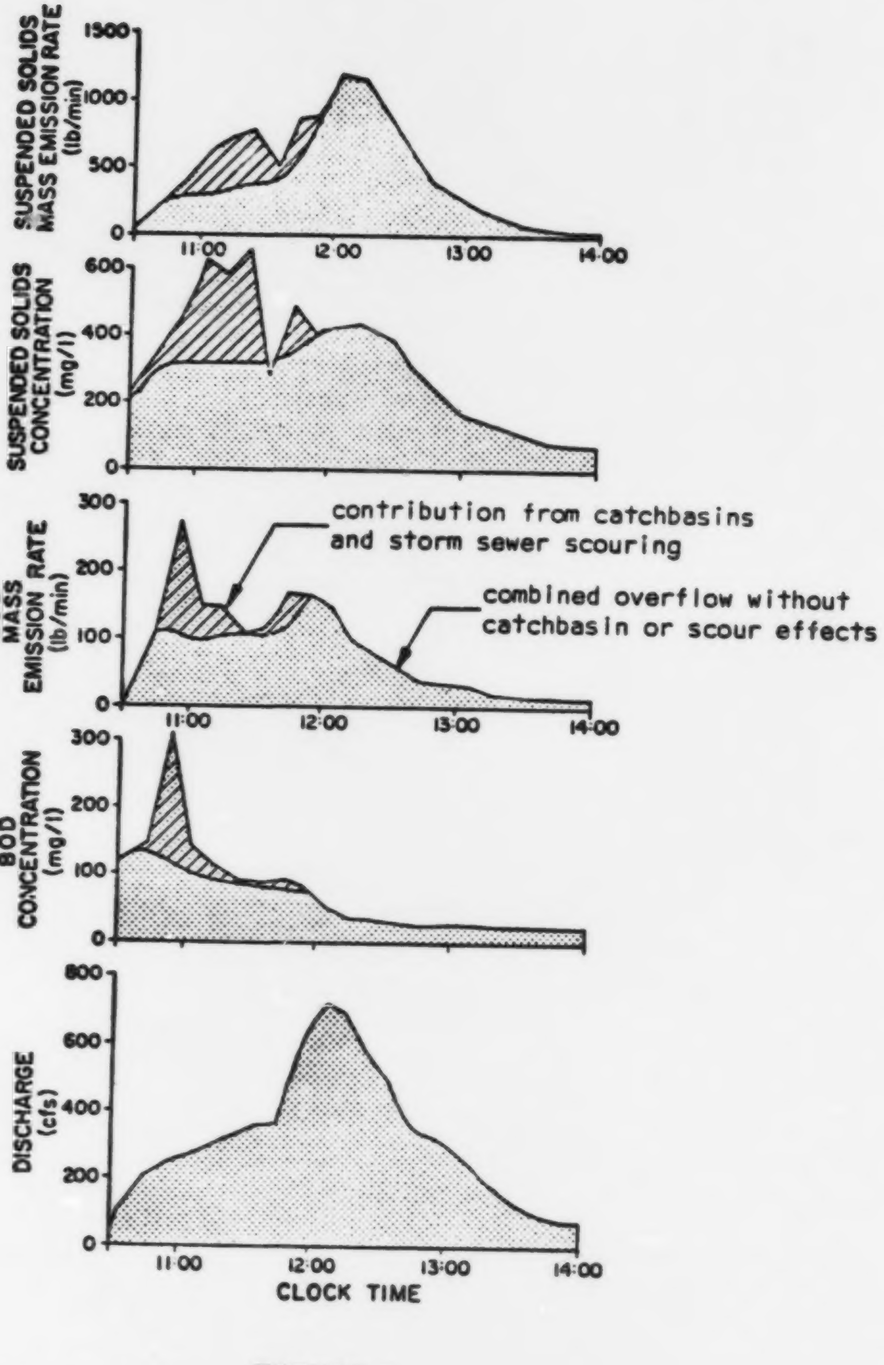

FIGURE 2
First-Flush Pollutional Effects of Catch Basins
And Sewer Scour on Combined Sewer Overflow
(Selby Street, San Francisco)

TABLE 3
Monthly Summary of Estimated Street Litter Components, From a 10-acre (4 ha) Residential Area, Chicago**

Month	Street Refuse Components (Tons/Month)					
	Rags	Paper	Dust & Dirt	Vegetation	Inorganic	Total*
Jan.	.0015	.036	.55	.00	.09	.68
Feb.	.0015	.036	.55	.00	.09	.68
March	.0015	.036	.55	.08	.09	.76
April	.0015	.036	.55	.08	.09	.76
May	.0015	.036	.55	.08	.09	.76
June	.0015	.036	.55	.08	.09	.76
July	.0015	.036	.55	.08	.09	.76
Aug.	.0015	.036	.55	.08	.09	.76
Sept.	.0015	.036	.55	.08	.09	.76
Oct.	.0015	.036	.55	.83	.09	1.56
Nov.	.0015	.036	.55	.83	.09	1.56
Dec.	.0015	.036	.55	.00	.09	.68
TOTAL*	.0180	.432	6.60	2.22	1.08	10.48

*Some totals have been rounded off.

It is readily apparent that the most significant component is dust and dirt except during the fall of the year when vegetation (primarily leaves) becomes the dominant component.

TYPES OF POLLUTANTS AND LOADING RATES

Nearly all of the pollutants found in urban runoff are associated with the dust and dirt component of street litter. By type, COD, BOD, and solids (suspended and settleables) are found in the greatest quantity. Nitrogen and phosphorus are also found in significant quantities. In areas where street deicing by salting is practiced, winter runoff contains very high chloride concentrations. Other pollutants found in urban runoff include pesticides, herbicides, fertilizers and other chemical additives, heavy metals, and many other known and unknown pollutants.

Data on the rate at which pollutants accumulate on an urban watershed is very scanty. In fact, it is almost non-existent. A lot of data has been collected on the quality of combined sewer overflows and stormwater discharges for various cities in the United States (Tulsa, Oklahoma; Washington, D.C.; Atlanta, Georgia; San Francisco, California; Sacramento, California; and Roanoke, Virginia) as a result of the U.S. Environmental Protection Agency's demonstration grants

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**This table is a reproduction of Table 4 in Reference 3.

program for abatement of stormwater pollution. The studies are reported in EPA's Water Pollution Research Series; however, the dissimilar forms in which the data is reported makes it difficult and in some cases impossible to generalize. The problem is that data is often presented as average concentrations or as pounds of pollutant runoff per inch of rain, and the reported values may be for combined wastewater and storm runoff rather than for storm runoff alone.

Even for a given watershed, there is no apt description of "typical" stormwater runoff characteristics because of the variablility of rainfall-runoff patterns. Thus, reports of "mean concentration" or pounds per inch of rainfall are meaningless as generalized variables and they show poor correlation with any runoff parameters of interest.

Results from a demonstration project conducted in Tulsa, Oklahoma [5], were summarized in terms of pounds of loading per day per mile of street for each of 15 areas sampled in the study. These results, presented in Table 4, give an indication of the magnitude of pollutant buildup for different land uses. These findings must be viewed with caution, however, because they were computed by taking the "average" concentration of the pollutant for all events monitored, which when multiplied by the total storm runoff gave "total annual mass emissions" converted into a rate per day per mile. A much more useful way to have developed this data would have been to sum the products of discharge and concentrations over each of the observed events and then to sum all such total storm emissions over a year.

One of the best existing sources of information on the rate of accumulation of pollutants on urban watersheds is that data collected in a field study in Chicago by APWA [3]. This study determined the rate of buildup of dust and dirt in the streets on a number of different test areas and then related the concentrations of various pollutants to the dust and dirt. A summary of their findings is contained in Table 5.

To convert the data contained in Table 5 into a form comparable with that of Table 4, the Dust and Dirt is multiplied by 2 (gutters per street) x 52.8 (100's of feet of gutter per mile) x (constituent concentration/1000). E.g., the rate of BOD accumulation on an urban area that is single family residential is: $0.7 \times (2 \times 52.8) \times 5.0/1000 = 0.36$ lbs/day/mile. Rates of pollutant buildup in pounds per day per mile are given in Table 6.

Comparison of the values in Tables 4 and 6 indicates that while the Chicago data is consistently lower than the Tulsa data, the rates of buildup are similar in commercial areas except for PO4. Multiple family residential values for the Chicago area also compare well with the Tulsa data except for PO4. The principal reason for the lower values in the Chicago data is that the reported values are for the soluble portion of the constituents and do not include that portion found in suspended and settleable material. Also, the Tulsa data are approximations based

TABLE 4
Average Daily Loads Per Mile of Street*
(Tulsa, Oklahoma)

Test Area	Total Street Miles	Average Load: lbs/day/mile of street				
		BOD	COD	Total Solids	Organic Kjeldahl Nitrogen	Soluble Orthophosphate
Residential						
3	14.87	1.41	11.46	120	0.26	0.34
5	16.32	2.80	21.43	43	0.11	0.13
7	6.84	1.20	7.20	63	0.12	0.10
8	6.97	2.72	20.89	69	0.12	0.21
9	3.11	1.12	13.09	47	0.07	0.11
11	49.05	1.60	13.29	66	0.08	0.15
13	5.58	2.58	15.16	81	0.25	0.20
15	2.06	2.47	8.67	56	0.07	0.17
Commerical						
2	7.41	2.54	15.12	92	0.32	0.29
10	12.99	2.10	20.44	82	0.16	0.13
12	3.39	4.53	25.47	113	0.22	0.30
Industrial						
1	11.46	4.85	41.10	838	0.41	1.30
4	28.40	3.98	29.29	175	0.28	0.30
6	12.24	1.70	12.73	49	0.09	0.13
Average Values						
Residential		1.98	13.9	63.1	0.14	0.18
Commercial		3.06	20.3	95.7	0.23	0.24
Industrial		3.51	27.7	354.	0.26	0.57

*Reproduced from Reference 5.

on over-simplified computations. Thus, of the two sets of reported data the Chicago data is probably better, given that the amount of constituents contained in the solids can be determined.

TABLE 5
APWA Findings on Rate of Pollutant Buildup On Urban Watersheds*

Amount of Dust and Dirt and Strength of BOD by Land Use

Land Use	Amt. of D/D by land use lb/day/100 ft of curb	BOD of D/D mg/g
Commercial	3.3	7.7
Industrial	4.6	3
Multiple family	2.3	3.6
Single family residence	0.7	5
Assumed weighted average	1.5	5

Amount of Pollutant by Type of Land Use

Item	Single Family	Multiple Family	Commercial
Water Soluble (mg/g)	6.0	5.6	12.4
Volatile Water Soluble (mg/g)	3.8	3.4	6.9
BOD (mg/g)	5.0	3.6	7.7
COD (mg/g)	40	40	39
PO_4 (mg/g)	.05	.05	.07
N (mg/g)	.48	.61	.41
Total plate counts/g (x 1000)	10,900	18,000	11,700
Confirmed coliform/g (x 1000)	1,300	2,700	1,700
Fecal enterococci/g	645	518	329

TABLE 6
Average Daily Loads Per Mile of Street
(Chicago, Illinois)

Land Use	Average Load: lbs/day/mile of street			
	BOD	COD	N	PO4
Single Family Residential	0.36	2.95	0.03	0.004
Multiple Family Residential	0.87	9.70	0.15	0.012
Commercial	2.70	13.6	0.14	0.024
Industrial	1.45			

One fact that is quite evident from both the Chicago and Tulsa data is that the rate of buildup of pollutants on an urban watershed varies significantly with land use. Intuition would tell us this is true. Both sets of data indicate that industrial and commercial areas are much dirtier than residential areas. This would be expected since there is higher pedestrian and vehicular traffic densities in these areas. The data shows that pollutant accumulation rates are approximately one and one-half to five times as great in commercial and industrial areas as they are in residential areas.

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*See Reference 3.

Additional information of pollutant accumulation rates can be found in Reference 6.

ENTRY OF POLLUTANTS INTO URBAN RUNOFF

The first raindrops that fall on an urban watershed simply wet the land surface. As additional rain falls, the impervious surface will become wet enough that some of the water begins to form puddles, filling the depression storage. This initial rain begins to dissolve the pollutants in the gutters, streets, and on other impervious surfaces and eventually, as this water actually begins to flow off the watershed, it carries the dissolved material in it.

As rainfall intensity increases, overland flow velocities become sufficient to pick up solids. Suspended solids are, of course, picked up at smaller velocities than settleable solids. The settleable solids are carried off the watershed in two ways. If the velocity is sufficiently high, the settleable solids may be suspended in the overland flow. At lower velocities, particles may simply be rolled along the bottom surface toward the stormwater inlet.

The rain that initially falls on pervious surfaces infiltrates into the ground. If the rainfall is sufficiently intense, the infiltration capacity may be exceeded and the excess rainfall begins to fill the depression storage on the pervious surfaces. Finally, if the rainfall is of sufficient intensity and duration, runoff will begin to flow off the pervious areas, onto the impervious areas and thence into the stormwater inlets. Present experience, however, indicates that the amount of runoff and hence the pollution loads contributed from pervious surfaces in urban areas , is small compared to that coming from the impervious areas and can be neglected in determining the quality of surface runoff. This is especially true of surfaces covered with vegetation such as lawns and gardens. Figure 3 illustrates the differences in runoff and pollution load from a watershed that would occur if it was converted from a park (90% pervious) into a multiple residential area (20% pervious).

ESTIMATION OF THE RATE OF POLLUTANT BUILDUP ON URBAN WATERSHEDS

Since dirt is the major component of street litter and is the primary source of pollutants in urban runoff, the most basic approach for estimating pollutant buildup rates would be to relate them to the dust and dirt accumulation rates.

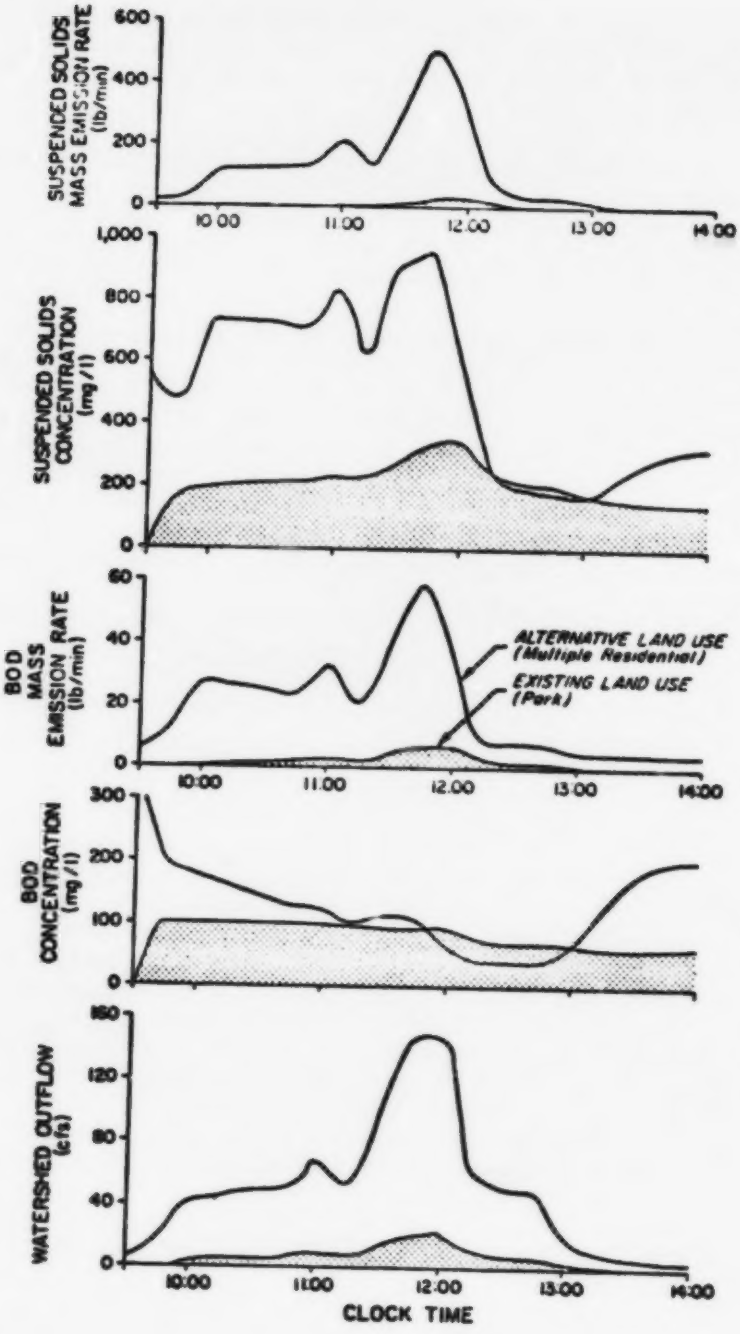


FIGURE 3
Effect of Changed Land Use on Characteristics
Of Subcatchment Runoff, Selby Street

Using APWA units (Table 5) for the rate of dust and dirt accumulation, the rate of buildup DD_L for a given land use L can be expressed as:

$$DD_L = dd_L \times (G_L/100) \times A_L \qquad (1)$$

where

DD_L = rate of dust and dirt accumulation on a watershed of land use L in lbs/day;

dd_L = rate of dust and dirt accumulation on watershed L in lbs/day/100 feet of gutter;

G_L = feet of gutter per acre in watershed L; and

A_L = area of watershed L in acres.

The initial quality of a pollutant p on watershed L at the beginning of a storm can then be computed as:

$$P_p = (F_p \times DD_L \times N_D) + P_{po} \qquad (2)$$

where

P_p = total pounds of pollutant p on the watershed L at the beginning of the storm;

F_p = pounds of pollutant p per pound of dust and dirt;

N_D = number of dry days since the last storm, and

P_{po} = total pounds of pollutant remaining on watershed L at the end of the last storm.

In practice, P_p is usually limited to the amount that would be accumulated in a 90-day dry period. The reason for this is that the efficacy of extrapolating daily buildup rates beyond this point (which was arbitrarily selected) is uncertain. Moreover, if equation (2) is used repetitively over long periods of time, positive errors could tend to accumulate in P_{po} resulting in overly large values of P_p.

If street sweeping is practiced on the watershed, the number of dry days since the last storm must be modified to account for the number of street sweepings that occurred since the last rainfall. The correct expression to use for P_p is then:

$$P_p = P_{po}(1-E)^n + N_S \times DD_L \times F_p \, [(1-E) + \ldots (1-E)^n] + DD_L \times F_p \, (N_D - n N_S) \qquad (4)$$

where

N_S = number of days between street sweepings;

n = number of times the street was swept since the last storm; and

E = efficiency of street sweeping (0.6 to 0.95).

DETERMINATION OF URBAN RUNOFF POLLUTION LOADS

To compute the amount of pollutant washed off the watershed during a storm, it is assumed that the amount of pollutant removed at any time t is proportional to the amount remaining:

$$\frac{dP_p}{dt} = -KP_p \tag{4}$$

We stated earlier that the runoff rate Q also affects rate of pollutant removal, therefore K must be functionally dependent upon Q. However, given two identical watersheds except for their area size, for the same rainfall rate r on both watersheds a higher runoff rate would occur from the larger watershed. This area effect can be eliminated by dividing the runoff Q by the impervious area of the watershed. The impervious area is used because only a negligible amount of the runoff comes from the pervious area. Since cfs per acre are equivalent to inches per hour, we can say that K is functionally dependent on the runoff rate R from the impervious area, where R is in inches per hour. Finally, assuming that K is directly proportional to R and that a uniform rainfall of 1/2 inch per hour would wash away 90 percent of the pollutant in one hour (a somewhat arbitrary assumption), we can say that K = 4.6R. Making this substitution into equation (4) and integrating over a time interval Δt (during which R is held constant) gives:

$$P_p(t + \Delta t) = P_p(t)e^{-4.6R\Delta t} \tag{5}$$

Equation (5) is the basic form of the overland flow quality model developed by Metcalf & Eddy, Inc., as part of the EPA Stormwater Management Model [7]. Although it is simplistic and contains many assumptions, it is the best overland flow water quality predictor or simulation model that presently exists. Moreover, experience with that model (See Reference 7) has shown it to give fairly good results.

Some idea of how equation (5) behaves can be gained by examination of Figures 4 and 5. Figure 4 shows that for a constant runoff rate R, the amount of pollutant remaining on the watershed decays exponentially. Under a time varying R the picture is quite different, as illustrated by the upper graph Figure 5. From the curve of P vs. t it can be seen that the amount of pollutant removed during an interval

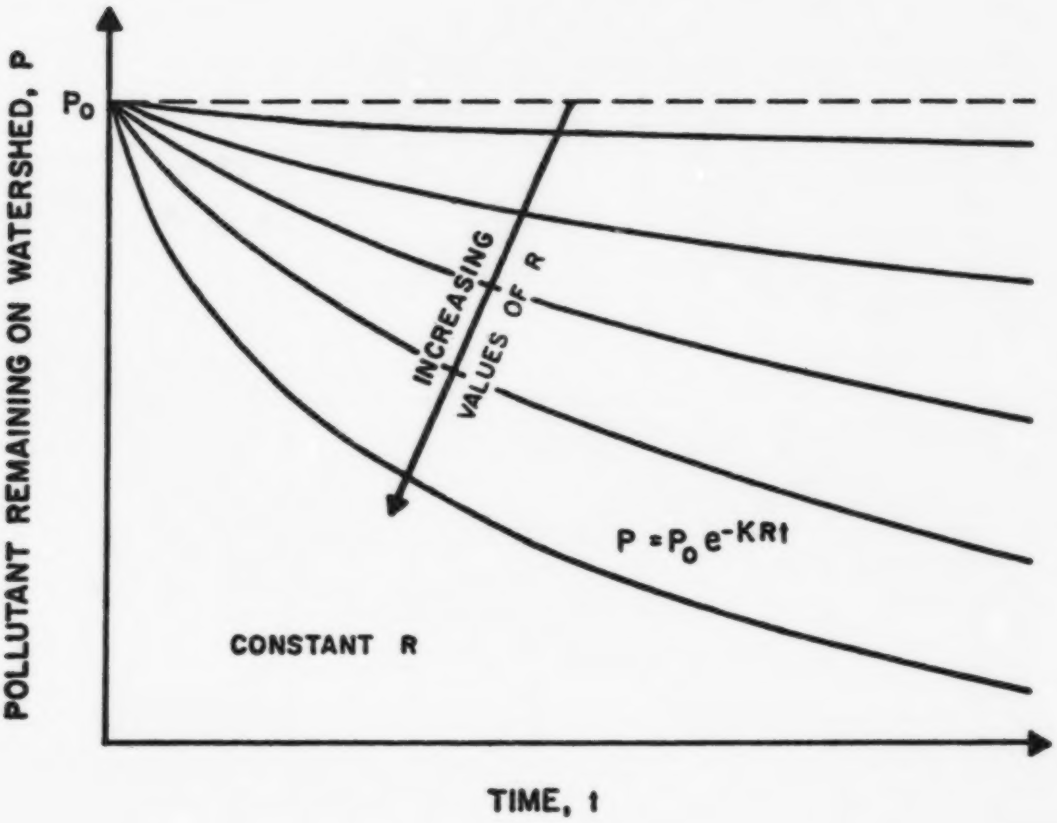


FIGURE 4
Basic Form of the Overland Flow Quality Model

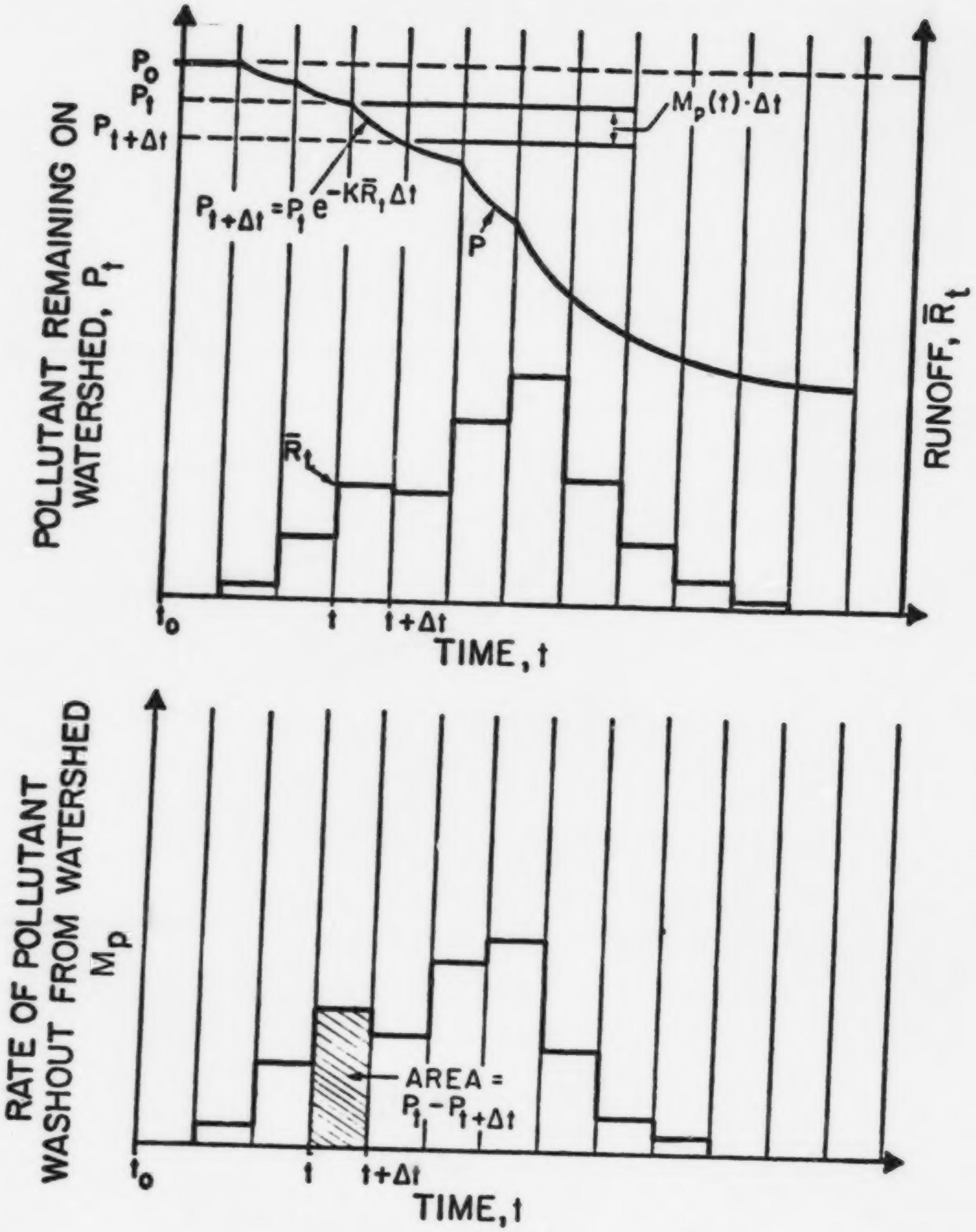

P_o
P_t
$P_{t+\Delta t}$
$M_p(t)\cdot\Delta t$
$P_{t+\Delta t}=P_t e^{-K\bar{R}_t\Delta t}$
P
$\bar{R}_t$
POLLUTANT REMAINING ON WATERSHED, P_t
RUNOFF, $\bar{R}_t$
t_o
t
t+Δt
TIME, t
RATE OF POLLUTANT WASHOUT FROM WATERSHED M_p
AREA = $P_t - P_{t+\Delta t}$
t_o
t
t+Δt
TIME, t

FIGURE 5
Development of Pollutograph (M_p vs. t)
from Time History of P_p

Δt is $P(t) - P(t+\Delta t)$. The rate of removal of mass from the watershed M_p is simply $[P(t) - P(t+\Delta t)]/\Delta t$, which can be expressed as:

$$M_p = P(t) \times (1 - e^{-4.6R\,\Delta t})/\Delta t \qquad (6)$$

The variation of M_p with time for the associated hydrograph is plotted in the lower graph of Figure 5. A plot of M_p versus t is termed a pollutograph, one of the most informative methods for expressing the pollutant load carried by urban runoff. To determine the concentration of a pollutant in the runoff as a function of time, one simply divides the pollutograph value M_p by Q (with appropriate conversion factors).

THE ROLE OF THE TRANSPORT SYSTEM

All of the preceding discussion has dealt with determination of the pollutant loads and quality of water washed off of the urban watersheds. Thus, the pollutographs shown in Figure 5, describe the rate of mass transport of a pollutant into the storm sewer system from a single watershed in an urban drainage area. For the watershed shown in Figure 6, there would be twenty such pollutographs formed. These individual pollutographs are then routed through the storm sewer or transport system using mass continuity to develop an outfall pollutograph at the lower end of the system. Depending upon the travel time in the transport system and the time to peak for the individual pollutographs, the resultant pollutograph at the outfall may have a high peak due to compounding of individual peaks from the tributary watersheds, or it may have a lower peak and broader base if the travel time in the sewer system is long compared to travel time on the individual watersheds.

The compounding effect is observed for stationary storms on steeper watersheds while the low-peak broad-base outfall pollutographs are observed in very flat systems. The compounding effect can also occur in a flat system however if the storm is moving along the fall line.

EFFECT OF LAND USE CHANGES ON STORM RUNOFF CHARACTERISTICS

To demonstrate the effect of land use changes on storm runoff characteristics, assume the park area in the Selby Street system (see Figure 6) was changed to a multiple residence area. The major significance of this change is to increase the impervious area of the subcatchment from 11 to 75 percent and to generate a dry weather sanitary flow from the area.

The effect of the modified land use on the characteristics of the runoff from the subcatchment are illustrated in Figure 7a. The effect of the increased imperviousness on the outflow hydrograph is dramatic, producing a seven-fold increase in peak runoff. The increased

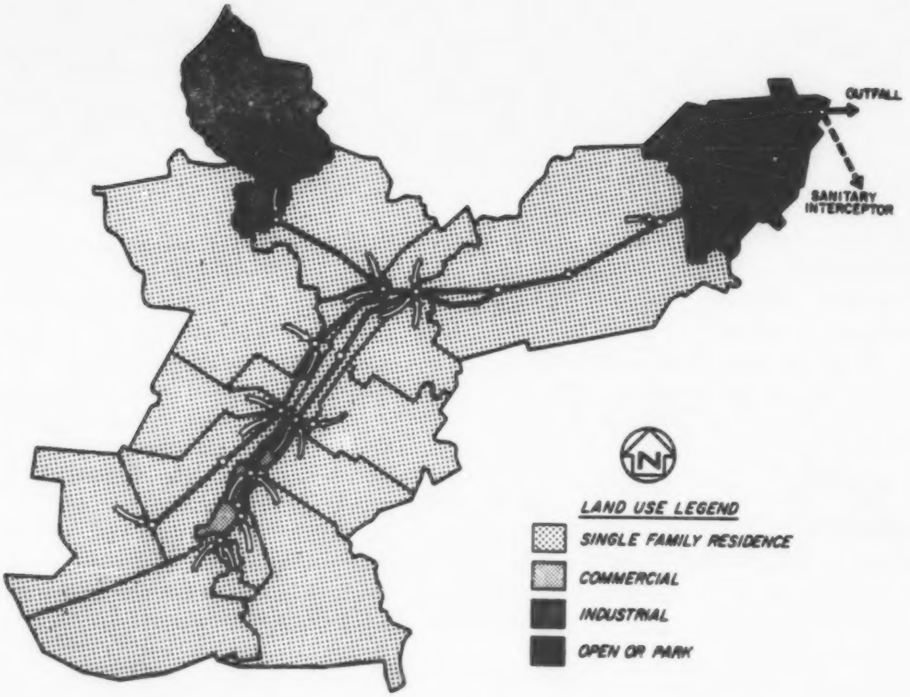
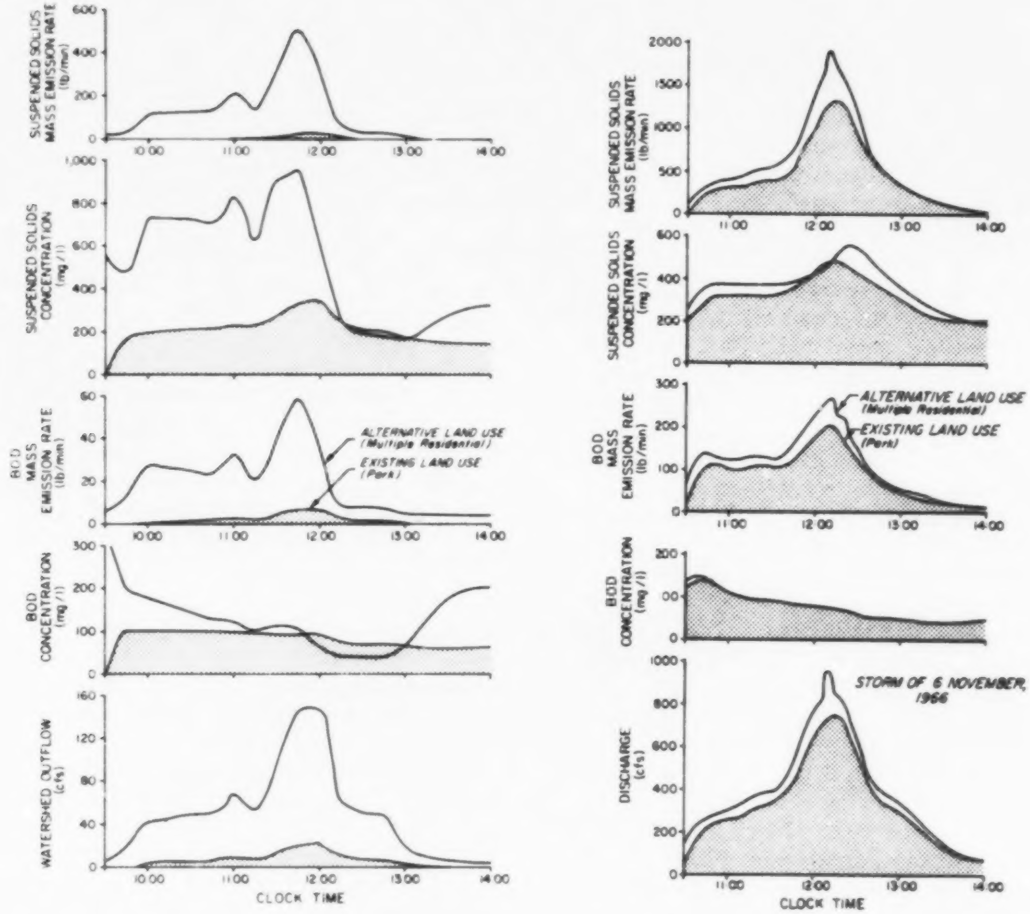

OUTFALL
SANITARY INTERCEPTOR
N
LAND USE LEGEND
SINGLE FAMILY RESIDENCE
COMMERCIAL
INDUSTRIAL
OPEN OR PARK

FIGURE 6
Watershed Subcatchments and Drainage System
Selby Street Drainage System



A. Effect of Changed Land Use on Characteristics of Watershed Runoff, Selby Street

B. Effect of Changed Land Use on Characteristics of Combined Sewer Overflow, Selby Street

FIGURE 7
Effects of Changed Land Use on Runoff Characteristics From Selby Street Watersheds

runoff rate scours additional surface pollutants off the watershed resulting in increased pollutant concentrations and mass emission rates. Notice that the mass emission rates for BOD and suspended solids decrease after the peak of the storm and remain approximately constant after 12:15. The concentrations of these pollutants in the watershed outflow, however, increase. This behavior is due to the influence of the dry weather flow and is explained as follows. By the time the surface runoff peaks, most of the surface pollutants have been washed off the watershed. Surface runoff after that time is rather clean and dilutes the dry weather sanitary flow coming off the subcatchment. Then as the surface runoff subsides further, the dilution effect disappears and the subcatchment outflow concentrations approach those of the dry weather flow.

The effect of the changed land use on the characteristics of the combined sewer overflow at the Selby Street outfall is illustrated in Figure 7b. As expected, the overflow discharge is increased as a result of the added runoff. The spike at the peak of the discharge curve is an interesting phenomena that results from the rapid peaking of the runoff from the multiple residential area. Recall that the imperviousness of this area is 1.7 times (75 percent versus 42 percent on the average) that of the remainder of the watershed; hence, the outflow from this subcatchment responds much more rapidly to changes in rainfall rates than to the other watersheds. The spike in the discharge overflow curve is caused by the rapid response of the multiple residential area to the rainfall increase at 11:30.

The indicated effect of the changed land use on the mass emission rates of BOD and suspended solids is expected. Notice that the mass emission rates for both of these pollutants reflect the effect of the surge in the overflow discharge. It is interesting to note that the BOD concentration in the overflow is essentially unchanged for the two cases even though the BOD load on the system is increased, as illustrated by the mass emission rate curve. This would indicate that the BOD concentration in the watershed outflow is not dramatically different for the case of the park and the multiple residential land use alternatives. Inspection of Figure 7b shows this to be true during the high runoff period.

The suspended solids concentrations for the multiple residential land use case are higher than for the park, and the peak concentration is lagged. The increased concentration is due to the increased wash off of surface pollutants as the result of the increased runoff. The reason for the lag in the peak concentration is not apparent.

CONCEPTS OF POLLUTION CONTROL OF URBAN RUNOFF

We will not deal with this subject at any length in this lecture since it will be dealt with in later lectures. However, some conceptual ideas should come to mind on the basis of the material presented here. To start with, we have seen that the major contribution to urban runoff pollution comes from the land surface itself. Hence, control of the pollution problem should start here. Many cities have begun anti-litter campaigns in commercial areas and have installed litter baskets at regular intervals at the curbside. These projects have been fairly successful and one need only look at the amount of trash in the containers to realize how much litter is being kept off the street as a result of these programs.

We have seen that dust and dirt is the principal component of street litter. Evaluation of pollution control alternatives should surely include a look at more frequent street sweeping, but not street flushing as is practiced in many cities since this simply places the pollution load farther down the system.

We have also seen that the principal source of runoff and pollution is from impervious areas. Figure 3 is a vivid example. One possible way to reduce runoff and pollution loads is to drain as much impervious area as possible to pervious areas. House gutters for example, should drain to lawns rather than the street or storm sewers. Parking lots could be made to drain into a gravel bed underneath the pavement before draining to the storm sewers system. This would provide temporary storage and a chance for the runoff to infiltrate (due regard must be taken in these cases, however, for possible contamination of groundwaters).

Another alternative is to collect stormwater in storage facilities and to treat it between storms. The proper combination of treatment and storage combined with some of the ideas outlined above would result in a very effective urban runoff management program from the standpoint of both quantity and quality.

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IMPACT OF STORMWATER RUNOFF ON RECEIVING WATER QUALITY

BY

Larry A. Roesner*

INTRODUCTION

At the present time, every city in the United States ultimately disposes of most of its stormwater runoff in a natural surface watercourse. The impact of these discharges on the quality of the receiving waters depends principally on three items:

1. The quantity and quality of the stormwater discharge(s).
2. The type of waterbody into which the discharge occurs.
3. Location of the outfall(s).

In the following section we will show the pollutional potential of stormwater and present some examples of its impact on different receiving waters. The last section contains general discussion on considerations involved in assessing the impact of stormwater discharges on receiving water quality.

POLLUTON POTENTIAL OF STORMWATER

Nearly every receiving waterbody has a set of water quality standards specified for it. These standards have generally been set on the basis of the "natural" quality of the water plus the beneficial uses identified for it. Table 1 lists water quality standards for three beneficial uses: drinking water supply, recreational use, and propagation of aquatic life. Comparison of these criteria show that the water quality standards vary significantly for different uses.

Some idea of the pollutional potential of stormwater runoff can be gained by examining Tables 2 and 3 which show measured concentrations of stormwater overflows in San Francisco, California and in Portland, Oregon, respectively. Note that in both cases the overflow qualities are for combined sewer overflows. Where applicable, water quality standards from Table 1 are also shown. It is evident from these data that a large pollution potential exists for untreated stormwater overflows with respect to such pollutants as suspended solids, COD, BOD, nitrogen, phosphorus, i.e. our standard pollutants. What is not shown, however, are the metals which are incorporated in the runoff and which pose a potential for toxicity effects to aquatic life in receiving waters. Table 4 shows recent combined sewerage data collected in Seattle, Washington. Here it can be seen that most of the metals concentrations approach the limit of the standards or exceed them. Iron and lead in particular both exceed the limit of the standards by nearly an order of magnitude.

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*Principal Engineer, Water Resources Engineers, Walnut Creek, California.

TABLE 1
Water Quality Criteria

CHEMICAL STANDARDS FOR DRINKING WATER (1)

Quality Factor	Recommended Maximum Limits,* mg/liter	Maximum Permissible** Concentrations, mg/liter
ABS (detergent)	0.5	
Arsenic	0.01	0.05
Barium		1.0
Cadmium		0.01
Carbon chloroform extract (exotic organic chemicals)	0.2	
Chloride	250.	
Chromium		0.05
Copper	1.0	
Cyanide	0.01	0.02
Fluoride	1.7	2.2
Iron plus manganese	0.3	
Iron	0.3	
Lead		0.05
Manganese	0.05	
Nitrate	45.	
Phenols	0.001	
Selenium		0.01
Silver		0.05
Sulfate	250.	
Total Dissolved Solids (TDS)	500.	
Zinc	5.	

*Concentrations in water should not be in excess of these limits, when more suitable supplies can be made available.
**"Maximum permissible" implies that which constitutes grounds for rejection of supply.

WATER QUALITY FOR AQUATIC LIFE (2,3)

Determination	Threshold concentration*	
	Freshwater	Saltwater
Total dissolved solids (TDS), mg/liter	2000†	
Electrical conductivity, mmhos/cm @ 25°C	3000†	
Temperature, maximum °C	34	34
Maximum for salmonoid fish	23	23
Range of pH	6.5-8.5	6.5-9.0
Dissolved oxygen (D.O.), minimum mg/liter	5.0‡	5.0‡
Floatable oil and grease, mg/liter	0	0†
Emulsified oil and grease, mg/liter	10†	10†
Detergent, ABS, mg/liter	2.0	2.0
Ammonia (free), mg/liter	0.5†	
Arsenic, mg/liter	1.0†	1.0†
Barium, mg/liter	5.0†	
Cadmium, mg/liter	0.01†	
Carbon Dioxide (free), mg/liter	1.0	
Chlorine (free), mg/liter	0.02	
Chromium, hexavalent, mg/liter	0.05†	0.05†
Copper, mg/liter	0.02†	0.05†
Cyanide, mg/liter	0.02†	0.02†
Fluoride, mg/liter	1.5†	1.5†
Lead, mg/liter	0.1†	0.1†
Mercury, mg/liter	0.01	0.01
Nickel, mg/liter	0.05†	
Phenolic compounds, as phenol, mg/liter	1.0	
Silver, mg/liter	0.01	0.01
Sulfide, dissolved, mg/liter	0.5†	0.5†
Zinc, mg/liter	0.1	

*Threshold concentration is the value that normally might not be deleterious to fish life. Waters that do not exceed these values should be suitable habitats for mixed fauna and flora.
†Values not to be exceeded more than 20 per cent of any 20 consecutive samples, nor in any 3 consecutive samples. Other values should never be exceeded. Frequency of sampling should be specified.
‡Dissolved oxygen concentrations should not fall below 5.0 mg/liter more than 20 per cent of the time and never below 2.0 mg/liter.

WATER QUALITIES FOR RECREATIONAL USE (2,3)

Determination	Water Contact		Boating and Aesthetics	
	Noticeable Threshold	Limiting Threshold	Noticeable Threshold	Limiting Threshold
Coliforms, MPN per 100 ml	1000*	†		
Visible solids of sewage origin	None	None	None	None
ABS (detergent), mg/liter	1*	2	1*	5
Suspended solids, mg/liter	20*	100	20*	100
Floatable oil and grease, mg/liter	0	5	0	10
Emulsified oil and grease, mg/liter	10*	20	10*	30
Turbidity, silica scale units	10*	50	10*	§
Color, standard cobalt scale units	15*	100	15*	100
Threshold odor number	32*	256	32*	256
Range of pH	6.5-9.0	6.0-10.0	6.5-9.0	6.0-10.0
Temperature, maximum °C	30	30	30	30
Transparency, Secchi disk, ft.	-	-	20*	§

*Value not to be exceeded in more than 20 per cent of 20 consecutive samples, nor in any 3 consecutive samples.
†No limiting concentration can be specified on the basis of epidemiological evidence, provided no fecal pollution is evident.
(Note: Noticeable threshold represents the level at which people begin to notice and perhaps complain. Limiting threshold is the level at which recreational use of water is prohibited or seriously impaired).
§No concentrations likely to be found in surface waters would impede use.

TABLE 2

Treatment of Combined Sewer Overflows, San Francisco (4)
Wet Weather Monitoring Results

Laguna Street Outfall* 15 March 1967

SAMPLE TIME	1 2020	2 2030	3 2045	4 2100	5 2115	6 2130	7 2215	8 2230	Standard from Table 1
FLOW, cfs	7.0	12.5	11.4	7.2	4.8	2.0	44	108	
COLIFORM MPN - Conf./Fecal, 10^4/ml	70/13	70/2.3	24/6.2	2.3/2.3	6.2/6.2	6.2/0.46	5/2.3	70/0.6	.01[1]
CONDUCTIVITY, umho/cm	338	220	200.8	178	206	219.8	158.3	76.8	3000. [2]
ALKALINITY, mg/l as $CaCO_3$	82	43.4	27.6	35.5	46.4	51.3	29.6	16.8	
SUSPENDED SOLIDS, mg/l	304	442	194	130	104	73	187	237	20. [2]
VOLATILE SUSPENDED SOLIDS, mg/l	234	264	149	86	81	56	92	119	
GREASE, mg/l	63.4	18.7	28.4	16.2	18.5	18.3	20.5	17.9	.2 [1]
COD, mg/l	458	425	210	198	169	251	210	165	
BOD, mg/l	252	169	97	104	92	108	51	41	
FLOATABLE PARTICULATES, mg/l	2.3	1.5	1.1	1.0	2.2	0.4	0.8	1.1	0. [2,3]
SETTLEABLE SOLIDS @ 30 Min., ml/l	40	35	13	7	12	10	5	15	
TOTAL KJELDAHL NITROGEN, mg/l -N	19.95	15.75	13.30	8.05	8.75	10.85	7.00	3.85	
AMMONIA NITROGEN, mg/l -N	10.50	4.20	2.80	3.50	4.20	4.55	1.40	2.45	
TOTAL PHOSPHATE, mg/l PO_4	3.20	2.12	1.47	1.41	1.86	1.40	1.17	1.13	
SULFATE, mg/l	29	18	15	18	17	19	17	10	250. [1]
CHLORIDE, mg/l	32	21	21	14	19.5	18	14	5.5	
SODIUM, mg/l	29.5	18.5	19.0	14.5	18.0	16.0	11.0	6.0	
POTASSIUM, mg/l	6.70	3.65	2.90	3.00	3.35	3.85	2.75	1.15	
CALCIUM, mg/l	11.2	10.4	8.0	8.4	8.0	8.8	7.2	4.8	
MAGNESIUM, mg/l	4.9	2.9	1.9	3.2	3.2	3.2	3.4	2.4	

*Land Use is Multiple Family Residential

[1] Chemical standards for Drinking Water

[2] Water Quality for Recreational Use

[3] Water Quality for Aquatic Life

TABLE 3
Summary of Characterization of Combined Sewage
Sullivan Gulch Pump Station (5) Portland, Oregon*
February - April 1969

CHARACTERISTIC	NUMBER OF OBSERVATIONS	MEAN	STANDARD DEVIATION	MINIMUM	MAXIMUM	Standard from Table 1
PH	26	5.0	+ .4	4.5	6.0	
TEMPERATURE, °F	25	48.7	+ 6.5	34.0	56.0	
DISSOLVED OXYGEN, MG/L	16	8.0	+ 2.2	3.7	10.4	5.[3]
SETTLEABLE SOLIDS, ML/L	25	3.1	+ 1.0	1.6	5.0	
TOTAL SUSPENDED SOLIDS, MG/L	28	146	+ 59	70	326	20.[3]
VOLATILE SUSPENDED SOLIDS, MG/L	28	80	+ 25	57	168	
% VOLATILE SUSPENDED SOLIDS	28	67	+ 17	38	93	
B.O.D., MG/L	14	105	+ 38	57	166	
C.O.D., MG/L	24	199	+ 50	138	324	
B.O.D./C.O.D.	14	.51	+ .08	.35	.64	
AMMONIA NITROGEN, MG/L	7	5.1	+ 1.4	3.7	7.0	
ORGANIC NITROGEN, MG/L	7	8.2	+ 3.1	5.10	14.0	
TOTAL NITROGEN, MG/L	7	13.3	+ 4.3	9.5	21.0	

*Land use is Single Family Residential

[1] Chemical standards for Drinking Water

[2] Water quality for Recreational Use

[3] Water quality for Aquatic Life

TABLE 4
Urban Runoff Characteristics
Viewridge Two Area (6)
Seattle, Washington**

Parameter	Feb 14	Mar 10	Mean Concentration Mar 16	June 6	Aug 16	Sept 19	Mean	Standard from Table 1
Temp. C^{o}	8.0	9.0	9.4	16.4	17.7	17.1	12.9	
pH	7.2	7.5	7.3	6.7	7.0	6.5	7.0	6.5-9.0^{3}
Cond. umho/cm	201	71	160	165	194	96	148	3000.^{3}
Turbidity, JTU	55	25	17.4	28.4	30.3	28.7	30.8	10.^{3}
DO, mg/l	10.3	11.5	10.5	6.6	6.8	8.2	9.0	8.^{3}
BOD, mg/l	9.3	16	5.8	39	100	12.4	30.4	
COD, mg/l	48	78	45	229	125	71	99	
Hexane Ext., mg/l	17.6	32	25.2	16.5	12.3	7.4	18.5	.2^{3}
Chloride, mg/l	22	2.0	4.8	17	21	3	11.6	
Sulfate, mg/l	25	5	24	20	28	9	20	250^{1}
Organic N, mg/l	-	-	.23	.66	6.41	1.41	1.71	
Ammonia N, mg/l	-	-	.16	.75	.27	.23	.35	
Nitrite N, mg/l	.19	.05	.09	.22	.15	.07	.13	
Nitrate N, mg/l	.73	.51	.51	1.38	.83	.49	.74	
Hydrolyzable P, mg/l	.20	.33	.14	.44	.81	.25	.36	
Ortho P, mg/l	.03	.04	.06	.22	.20	.12	.11	
Copper, mg/l	.034	.034	.031	.046	.001	.21	.059	.02^{3}
Lead, mg/l	.43	.094	.38	.37	-	.51	.36	.05^{1}
Iron, mg/l	3.4	.44	3.46	1.02	-	1.62	1.99	.3^{1}
Mercury, mg/l	-	-	.00020	-	-	.00014	.00017	.01^{3}
Chromium, mg/l	.0050	.0010	.0010	.010	.0074	.010	.0072	.05^{1,3}
Cadmium, mg/l	.0044	-	.057	.0045	.0031	.004	.015	.01^{1,3}
Zinc, mg/l	.055	.021	.030	.25	-	.24	.12	.1^{3}
Sett. Solids mg/l	136	.33	42	83	371	62	121	
Susp. Solids, mg/l	235	132	36	79	396	80	160	20^{2}
TDS, mg/l	154	98	151	199	181	78	144	500^{1}
Tot. Coliform* org./100 mls	7200	11000	6800	40000	42000	620000	26000	1000^{2}
F. Coliform* org./100 mls	450	2000	480	360	6000	13000	1200	

*Medians

**Land use is Single Family Residential

[1] Chemical Standards for Drinking Water

[2] Water quality for Recreational Use

[3] Water quality for Aquatic Life

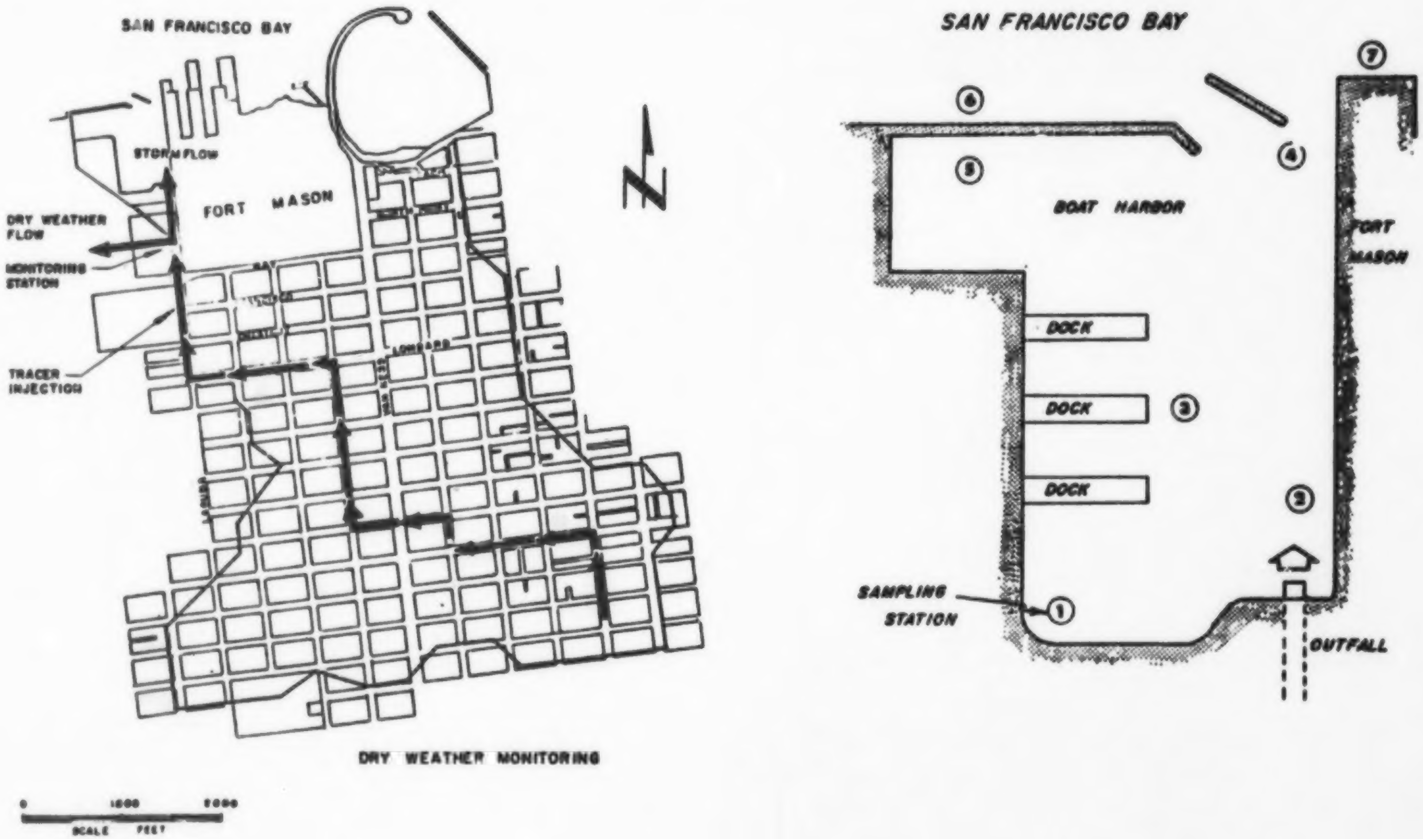
STORMWATER POLLUTION OF RECEIVING WATERS

The actual degree of pollution that occurs in a receiving water due to a stormwater discharge depends to a large extent upon the type of receiving water and the location of the outfall. Consider, for example, the Laguna Street confined sewer outfall in San Francisco. Figure 1 shows the drainage area (primarily middle class multiple family housing) plus an enlarged plan view of the municipal marina into which the Laguna Street outfall discharges. The marina waters are designated as recreational. Figure 2 shows confirmed coliform MPN's measured within the harbor and outside the harbor during the period 11-18 March 1967. It is readily apparent from the figure that there is significant pollution both inside and outside the harbor as a result of the Laguna Street stormwater overflows.

To illustrate the effect of outfall location on the dispersion characteristics of the receiving waters, consider Figure 3 which identifies the location of the Baker Street combined sewer outfall in San Francisco. This outfall although located less than one mile east of the Laguna Street outfall, produces an entirely different receiving water response because the outfall discharges to the open shoreline rather than an enclosed marina. Thus, concentrations around the outfall are significantly affected by the strong tidal currents through the Golden Gate. For the storm of 19-20 December 1969, Figure 4 shows the rainfall hyetograph, and the hydrograph and pollutographs at the Baker Street outfall. Figure 5 shows the mathematically simulated boundary of the pollutant field at 3, 4, and 5 hours after the start of rainfall. The effect of tidal influence on movement of the pollutant field is clearly evident in the figure. The effect of mixing and dispersion can be seen in Figure 6 which shows time histories of suspended solids in the vicinity of the outfall. Comparison of Figures 2 and 6 show that, on the average, Baker Street discharges are diluted two to five times more than those in the enclosed Laguna Street Marina.

As a last example, we consider a discharge of a combined overflow to a stream. Figure 7 shows the Kingman Lake drainage area in Washington, D.C. The Anacostia-Potomac Rivers were modeled by the 47 node system shown in the figure. An appropriate tide was imposed at node 32, freshwater inflows were imposed at nodes 1 and 47, and the Kingman Lake basin discharge for the storm of 22 July 1969 (see Table 5) was imposed at node 15. The computed oxygen balance in the receiving water system for 5 and 25 hours following the storm of 22 July is shown in Figure 8. The maximum deficit occurred at node 17, 3,000 feet from the point of release and 25 hours after the start of the storm. The oxygen deficit was continuing to increase and move seaward at the end of the simulation. Similarly, the computed travel of suspended solids is shown in Figure 8, also.

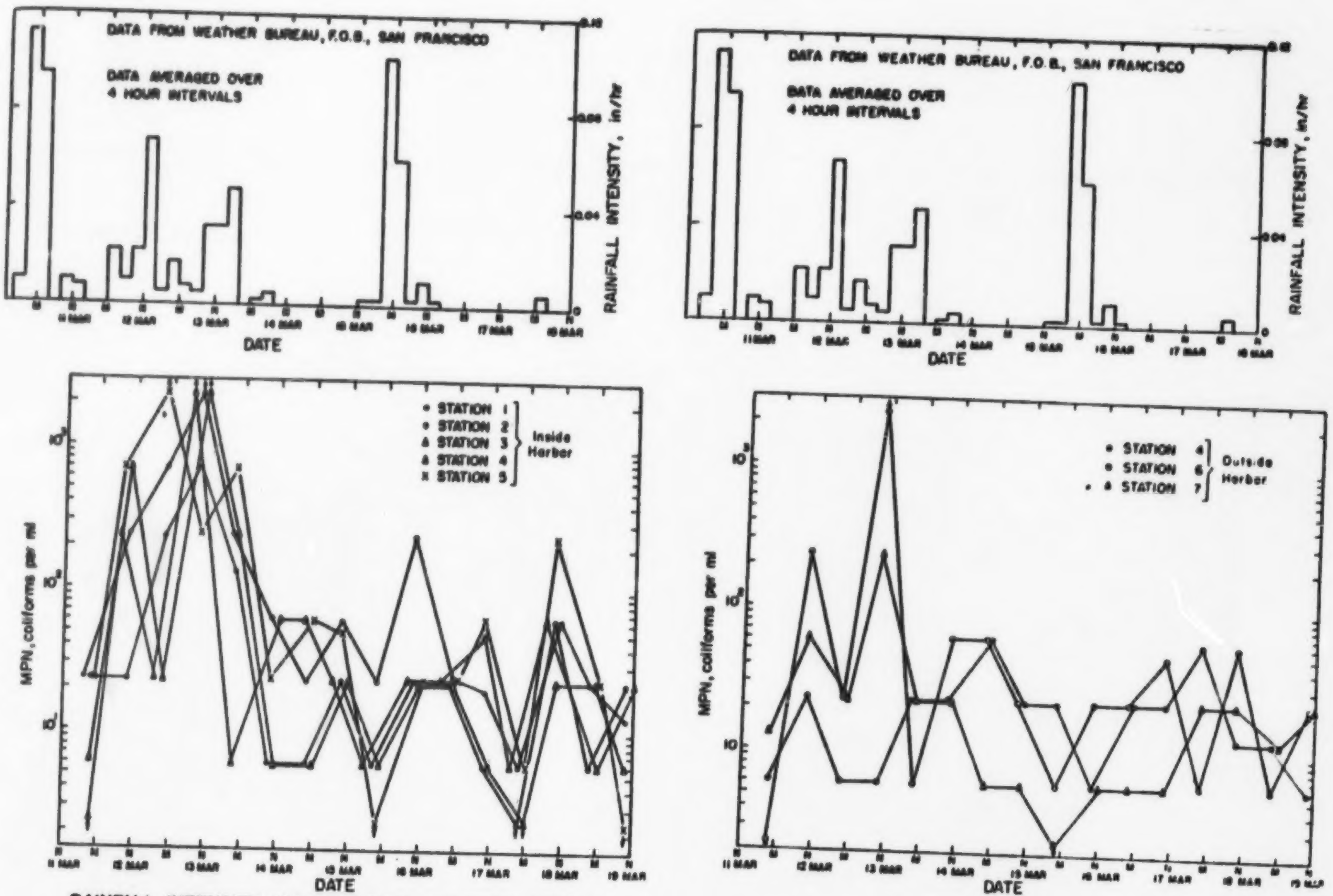
It should be noted that these changes were the direct result of one outfall discharging for one very large storm. The residual effects from earlier storms and pollution releases from coincident discharges would have to be evaluated to determine the full impact on the river system.


SAN FRANCISCO BAY
STORM FLOW
FORT MASON
DRY WEATHER FLOW
MONITORING STATION
TRACER INJECTION
DRY WEATHER MONITORING
0
1000
2000
SCALE FEET
SAN FRANCISCO BAY
⑥
⑤
④
⑦
BOAT HARBOR
FORT MASON
DOCK
DOCK
DOCK
③
②
SAMPLING STATION
①
OUTFALL

LAGUNA STREET OUTFALL DRY WEATHER MONITORING

LAGUNA STREET
RECEIVING WATER SAMPLING STATIONS

FIGURE 1 Laguna Street Drainage Area and Receiving Water Marina
(Reproduced from Reference 4)



RAINFALL INTENSITY AND CONFIRMED COLIFORM MPN'S IN LAGUNA OUTFALL RECEIVING WATERS - MARCH 1967

RAINFALL INTENSITY AND CONFIRMED COLIFORM MPN'S IN LAGUNA OUTFALL RECEIVING WATERS - MARCH 1967

FIGURE 2 Coliform Counts Measured in Laguna Street Marina
(Reproduced from Reference 4)

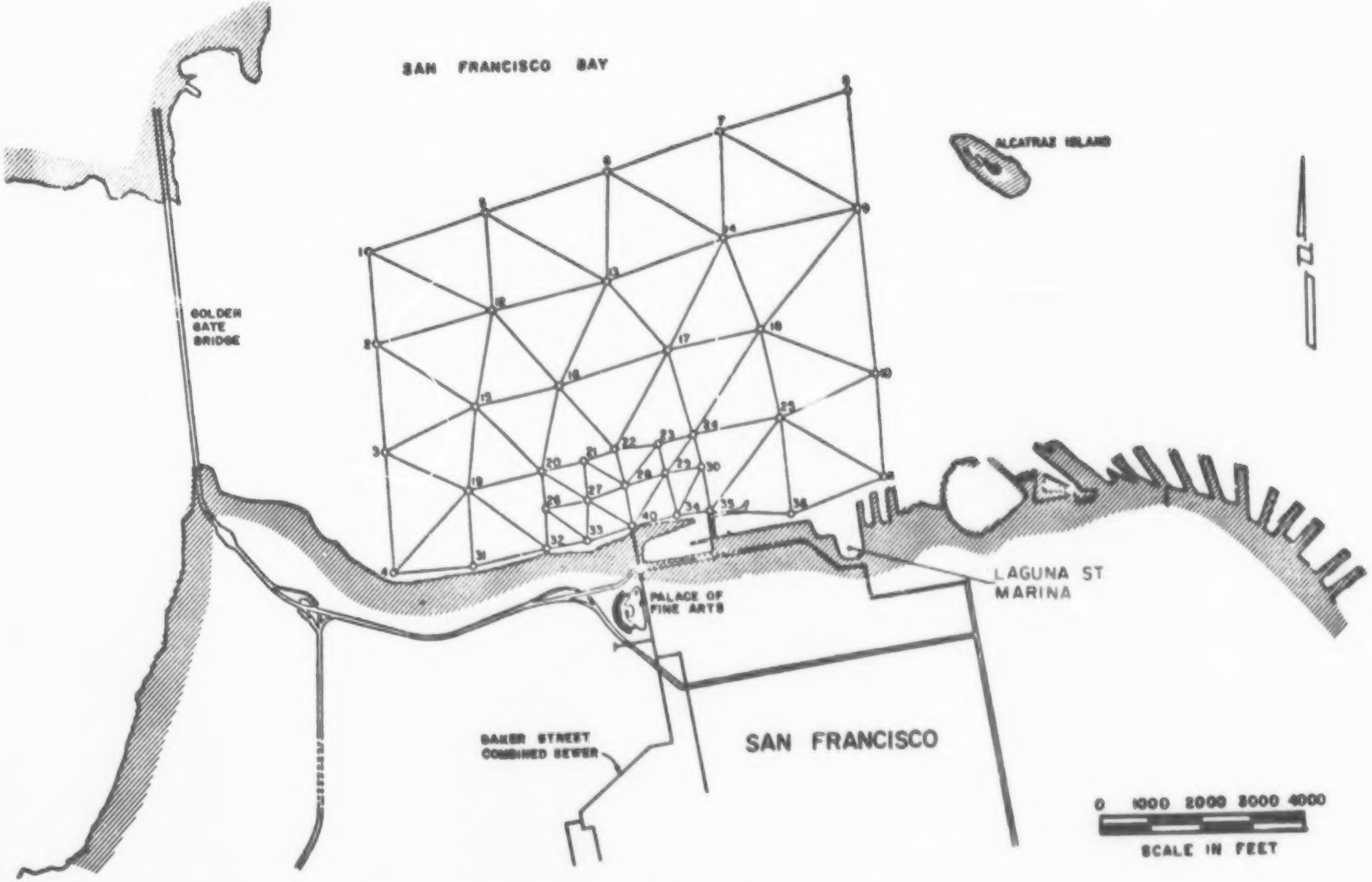

SAN FRANCISCO BAY
ALCATRAZ ISLAND
GOLDEN GATE BRIDGE
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
40
PALACE OF FINE ARTS
LAGUNA ST MARINA
BAKER STREET COMBINED SEWER
SAN FRANCISCO
0 1000 2000 3000 4000
SCALE IN FEET

FIGURE 3 Baker Street Receiving Water Grid System
(Reproduced from Reference 7)

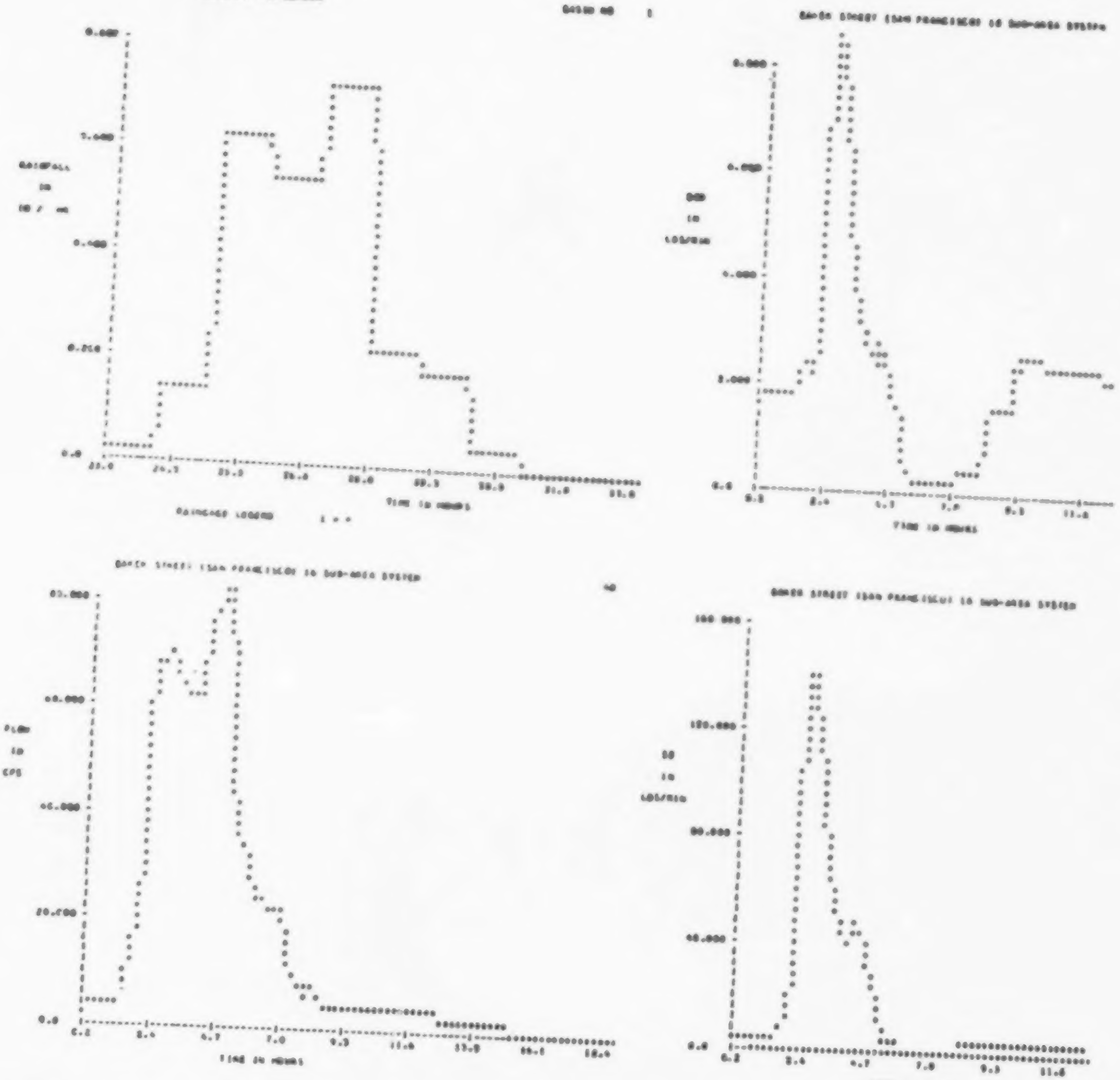


FIGURE 4 Baker Street Combined Sewer Overflow Results
Storm of December 19-20, 1969
(Reproduced from Reference 7)

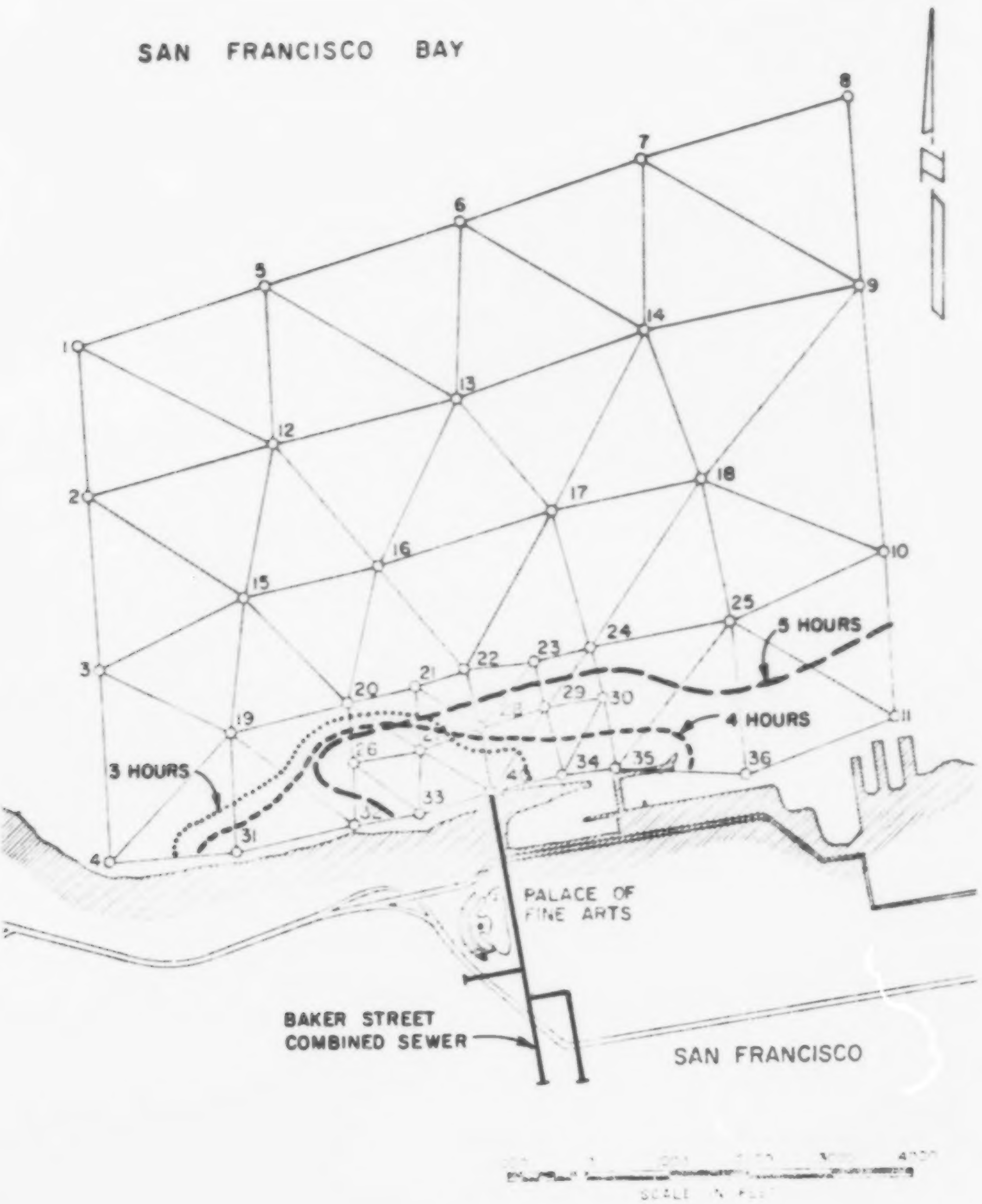

SAN FRANCISCO BAY
N
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
29
30
31
32
33
34
35
36
5 HOURS
4 HOURS
3 HOURS
PALACE OF FINE ARTS
BAKER STREET COMBINED SEWER
SAN FRANCISCO
SCALE IN FEET

FIGURE 5
Baker Street Receiving Water BOD Movement
(Reproduced from Reference 7)

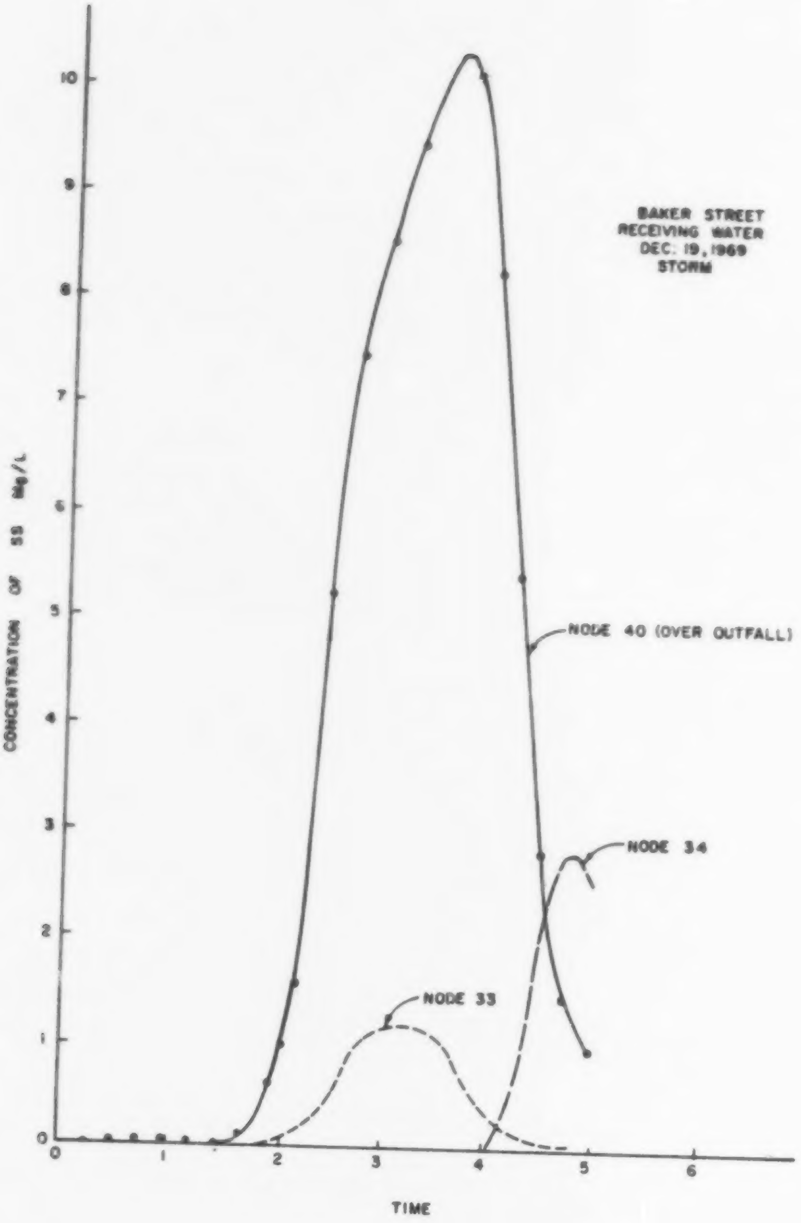


FIGURE 6
Concentration History at Three Junction Points
(Reproduced from Reference 7)

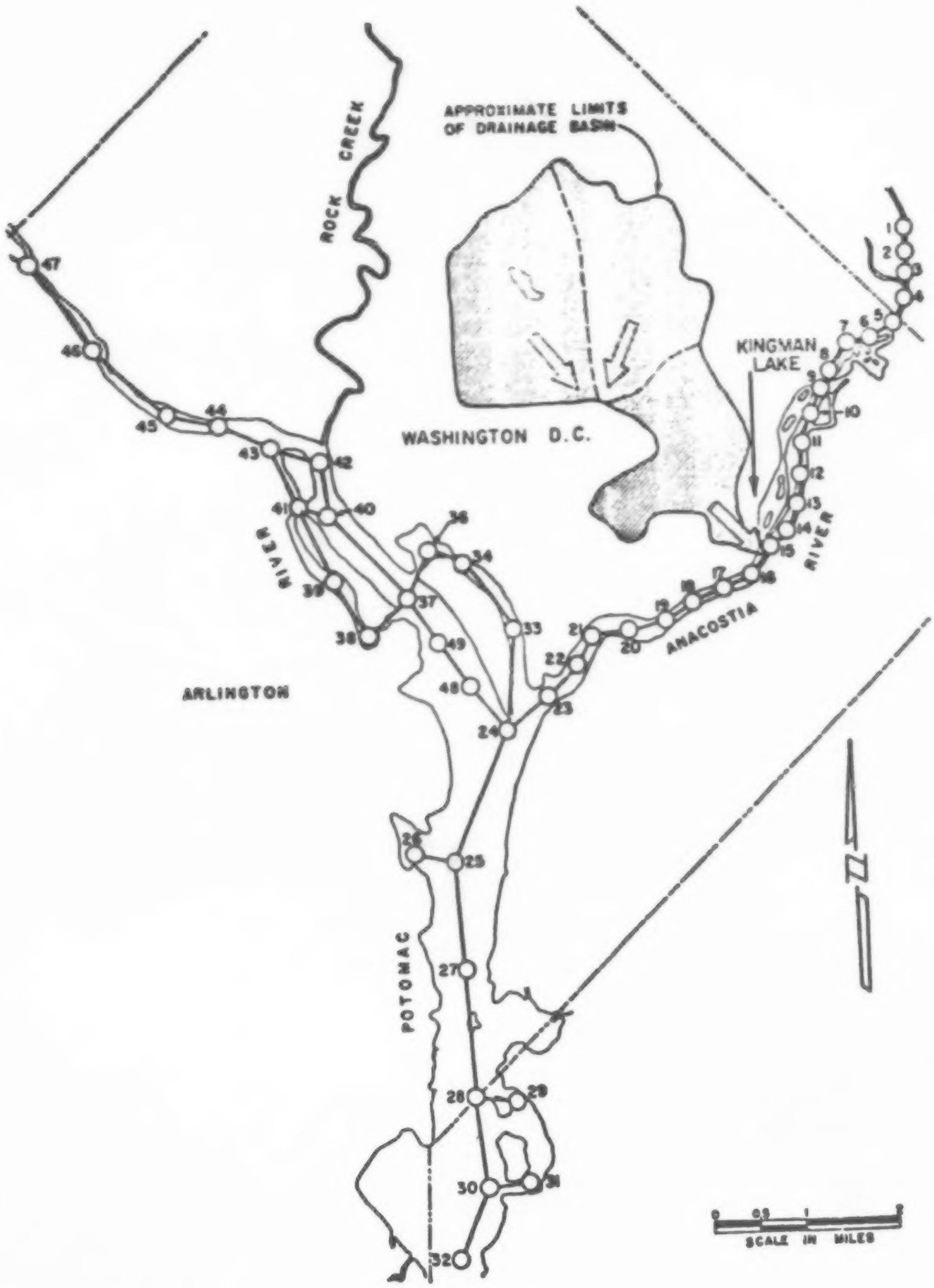
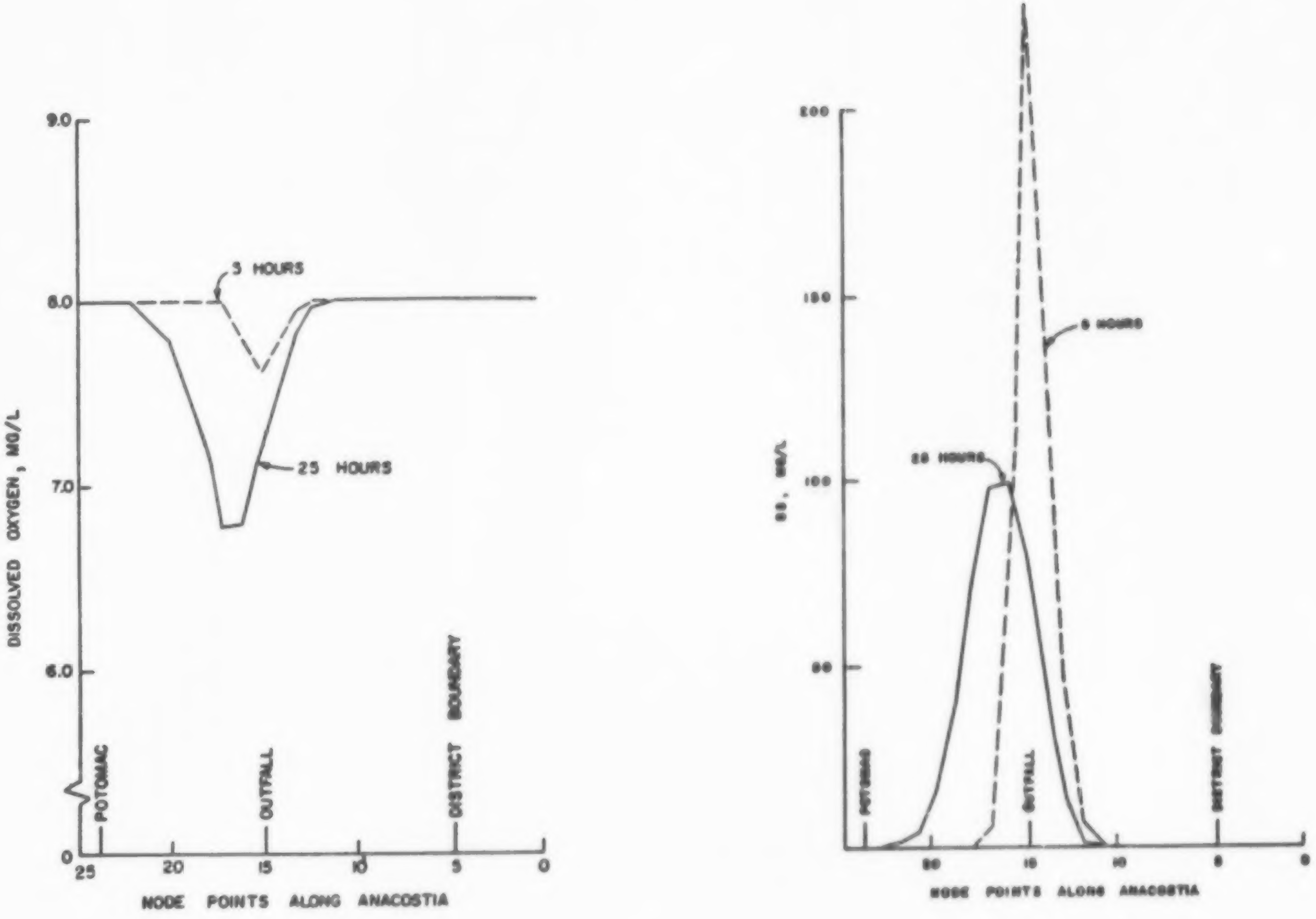

APPROXIMATE LIMITS OF DRAINAGE BASIN
ROCK CREEK
WASHINGTON D.C.
KINGMAN LAKE
RIVER
ANACOSTIA
RIVER
ARLINGTON
POTOMAC
N
0
0.5
1
2
SCALE IN MILES
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
36
37
38
39
40
41
42
43
44
45
46
47
48
49

FIGURE 7
Kingman Lake Receiving Water System
(Reproduced from Reference 7)

TABLE 5
Computed Time Variation of Overflow Quality
(Reproduced from Reference 7)

Time from Start of Overflow, min	Storm of July 22, 1969		Storm of Aug. 20, 1969	
	BOD, mg/L	SS, mg/L	BOD, mg/L	SS, mg/L
0	220	261	245	321
30	135	425	239	327
60	79	921	218	289
90	54	755	150	277
120	48	798	87	506
150	40	698	70	675
180	14	252	60	779
210	4	25	54	576
240	6	9	55	518
270	6	7	47	450
300	10	13	43	410
330	18	20	47	325
360	27	28	58	241
390	37	38	67	201
420	42	44	72	179
450	63	60	96	175
480	90	80	125	180



KINGMAN LAKE RECEIVING WATER
DISSOLVED OXYGEN PROFILE

KINGMAN LAKE RECEIVING WATER SS PROFILE

FIGURE 8 Kingman Lake Receiving Water Response to Storm of 22 July 1969
(Reproduced from Reference 7)

ENVIRONMENTAL ASSESSMENT CONSIDERATIONS

Receiving waters serve multiple uses.; thus procedures established for the planning of multiple purpose projects apply. We cannot simply chlorinate wastewater to maintain low coliform counts on a beach and, in turn, introduce a level of toxicity that will kill fish. Nor can we channelize the river for navigation or flood control without giving due consideration to the habitat of fish and fish-food organisms.

Figure 9 shows a conceptual diagram of an ecologic-water quality model. The interactions shown by the arrows are ecologic processes that transform chemicals such as carbon, nitrogen and phosphorus between their abiotic state and the successions of organic biomass. It is apparent from this figure that if the water quality is changed as the result of a waste stormwater discharge, there will be a resultant shift in the ecologic balance of the system. The severity and duration of the shift can be directly related to the severity and duration of the discharge.

The bases for environmental assessment are computations and value judgments. Computations are performed according to established knowledge or theory. Value judgment is then applied to extend the assessment beyond the state of the art of current computational technology.

Computation analysis should be carried out to depict effects with time and spatial detail. Along the time axis, there are long-term and short-term effects of urban runoff. In the spatial scale, the effect may be regional, i.e., the effect may be detected elsewhere downstream. Thus, stormwater runoff may create a transient increase in suspended solids and bacterial counts. Bacteria may die off rapidly but suspended solids may settle,exert a long-term effect on the ecosystem.

The extent to which value judgment must be applied to the assessment of ecologic impact depends on the degree of sophistication used in the computational analysis. The version of RECEIV documented in the EPA STORMWATER MANAGEMENT MODEL (8) simulates quality effects of BOD, dissolved oxygen, and suspended solids only. All other ecologic-water quality impacts must be inferred by value judgments. This model has recently been expanded, however, in the EPA's River Basins Modeling Projects program to include nine water quality parameters plus algae (9). Use of this model requires value judgments for evaluation of impact on higher trophic levels plus benthos.

Perhaps the most comprehensive computational tool presently available for ecologic-water quality assessment is the so-called Ecologic Model (10), which describes the interrelationship between some 23 water quality constituents and four biotic trophic levels. It too, however, requires the use of value judgments for interpretation of computed results and implication of these results on factors and processes that are not adequately accounted for in the models.

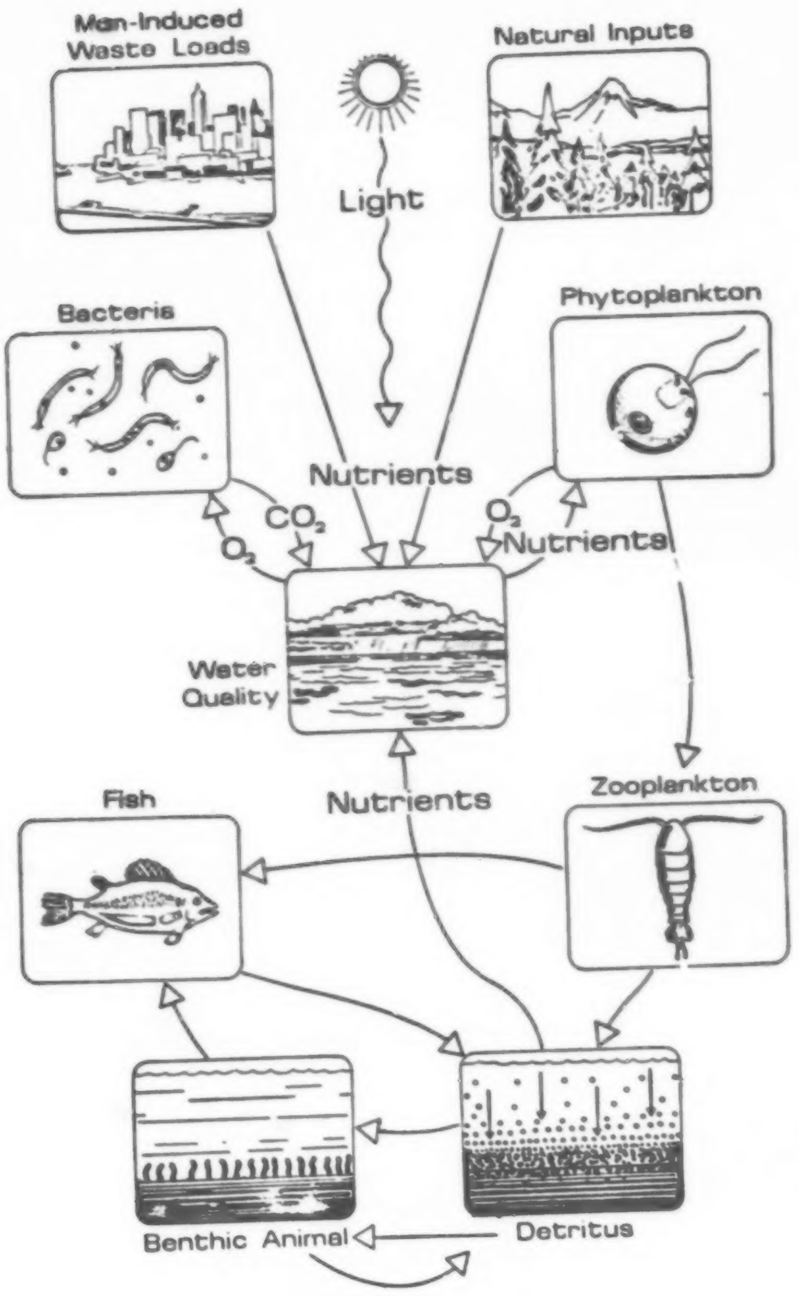

Man-Induced Waste Loads
Natural Inputs
Light
Bacteria
Phytoplankton
Nutrients
CO_2
O_2
O_2
Nutrients
Water Quality
Nutrients
Fish
Zooplankton
Benthic Animal
Detritus

FIGURE 9
Conceptual Diagram of Ecologic-Water Quality System

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4. Engineering Science, Inc., Characterization and Treatment of Combined Sewer Overflows, Final report submitted by Department of Public Works, City and County of San Francisco to FWPCA on Grant WPD-112-01-66, November 1967.

5. Marske, D.M., "Rotary Vibratory Fine Screening of Combined Sewer Overflows," Section 2 of Combined Sewer Overflow Abatement Technology, Water Pollution Control Research Series 11024-16/70, June 1970.

6. Farris, G., R.G. Swartz, and N.R. Wells, "Water Quality and Quantity Monitoring for the Ribco Urban Runoff and Basin Drainage Study," Draft Report, Municipality of Metropolitan Seattle, Seattle, Washington, April 1974.

7. "Storm Water Management Model, Vol. II, Verification and Testing," 11024 DOC 08/71, Metcalf and Eddy, Inc., Univ. of Florida, Gainesville, and Water Resources Engineers, Inc., Report for EPA, August 1971.

8. "Storm Water Management Model, Vol. I, Final Report," 11024 DOC 07/71, Metcalf and Eddy, Inc., Univ. of Florida, Gainesville, and Water Resources Engineers, Inc., Report for EPA, July 1971.

9. Water Resources Engineers, Computer Program Documentation for the Dynamic Estuary Model, An Intermediate Technical Report, EPA Contract NO. 68-01-1800, May 1974.

10. Chen, Carl W. and G.T. Orlob, "Ecologic Simulation for Aquatic Environments," Final Report to the Office of Water Resources Research, Water Resources Engineers, December 1972.

INTRODUCTION TO URBAN STORMWATER RUNOFF MODELS

By

Robert F. Shubinski*

DEFINITION OF THE URBAN DRAINAGE SYSTEM

The succeeding sections will be concerned with an **environment** that is predominantly urban. That is, it will be characterized by a high proportion of impervious (or nearly impervious) surfaces. Moreover, it is a system that will include man-made impervious pathways for guiding the flow of water over the surface (curbs, gutters, lined channels, paved parking areas, streets, etc.) and underground (storm, wastewater and combined sewers). The system includes all appurtenances that guide, control, or otherwise modify either the quantity, rate of flow or quality of runoff from urban drainage such as catch basins, storage basins, inlets, manholes, sediment traps, wiers and outfall structures.

Figure 1 illustrates a somewhat simplified urban drainage system. In this case, we view the system as an assemblage of subsystems dealing with

a. surface runoff
b. transport of flow and quality, and
c. the receiving water.

SURFACE RUNOFF SUBSYSTEM

The Surface Runoff Subsystem is illustrated for our example in Figure 2 which depicts the drainage area tributary to a sewer inlet as a system of surface elements (the rectangles), gutters (the dotted line) and drainage ditches (the dashed line). Each drainage subarea is characterized by its area, a degree of imperviousness, its slope, and certain coefficients that relate to its production of quality constituents that may be transported to the inlet by overland flow.

Input to the subsystem is comprised of rainfall that may be described in terms of an intensity-time graph derived from direct measurements in the watershed. The inset in the upper left of Figure 2 illustrates a typical rainfall hyetograph.

Within the subsystem a certain mass of a quality constituent (pollutant) may exist at the outset of the storm and be delivered by the flow at mass rates and concentrations that may depend on the nature of the storm, the character of the surface, and the sources of the pollutant (upper right insert, Figure 2).

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*Principal Engineer, Water Resources Engineers, Springfield, Virginia.

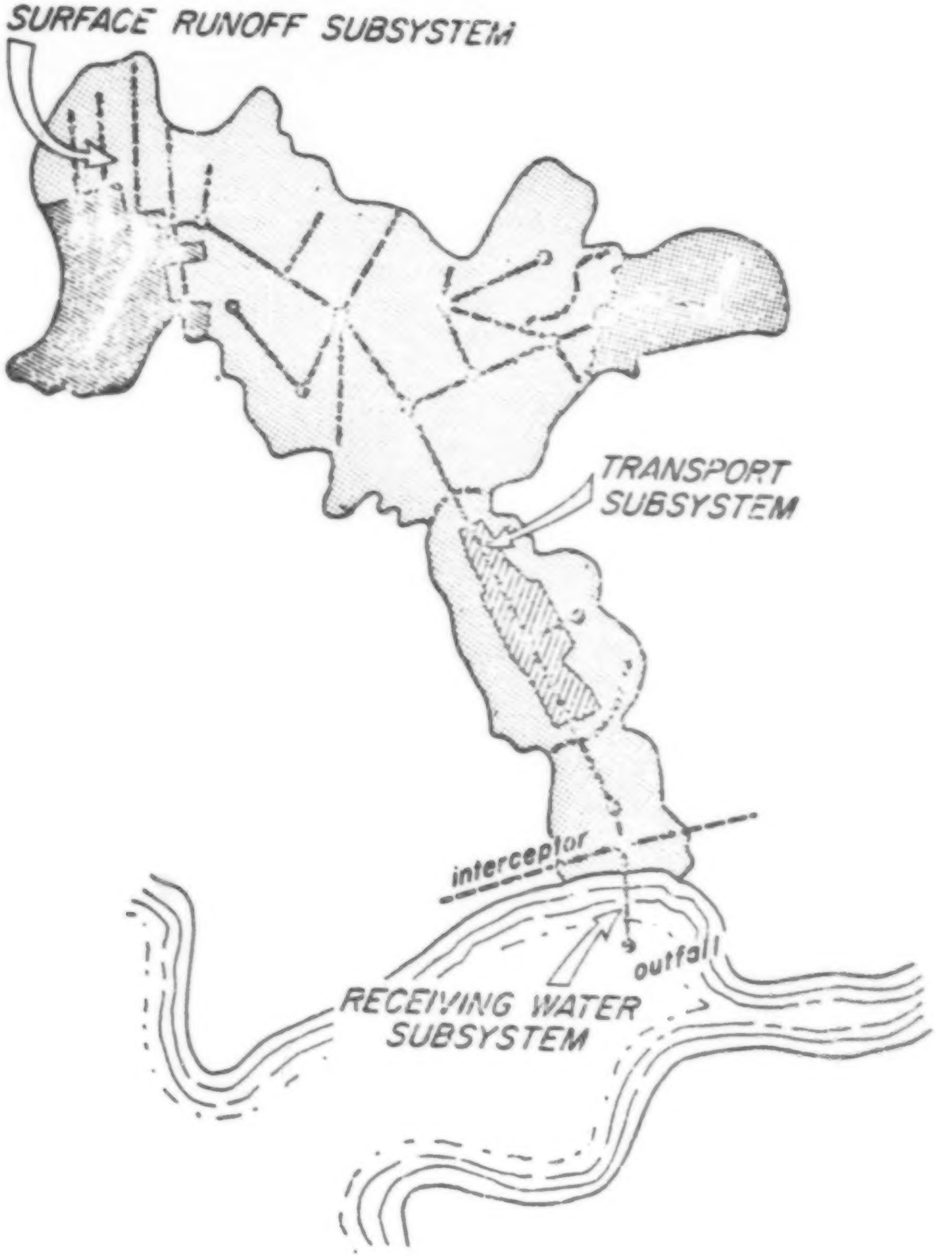
SURFACE RUNOFF SUBSYSTEM
TRANSPORT SUBSYSTEM
interceptor
outfall
RECEIVING WATER SUBSYSTEM

FIGURE 1. THE URBAN DRAINAGE SUBSYSTEM

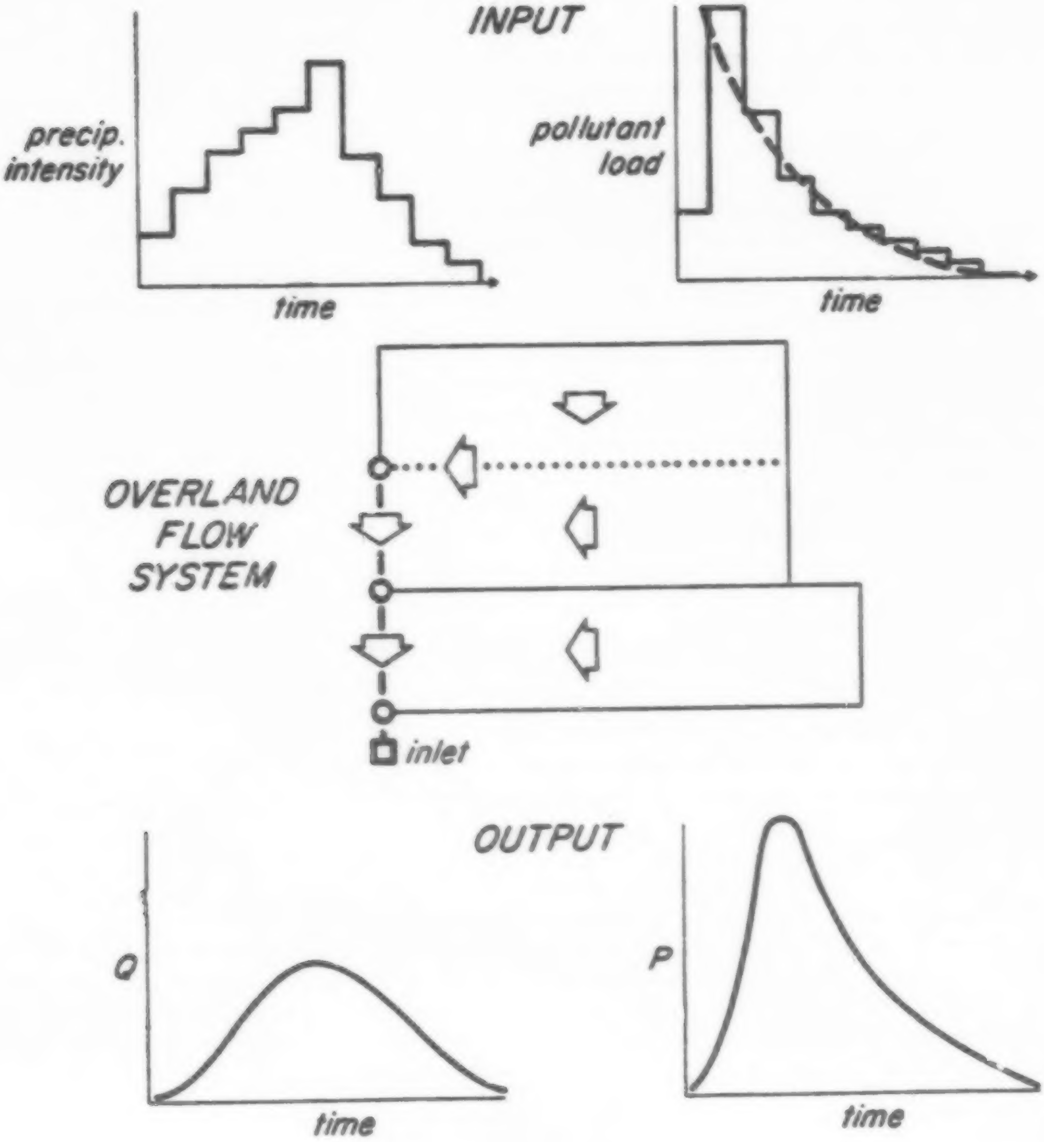
INPUT
precip. intensity
time
pollutant load
time
OVERLAND FLOW SYSTEM
inlet
OUTPUT
Q
time
P
time

FIGURE 2. SURFACE RUNOFF SUBSYSTEM

The overland flow process modifies the rainfall hyetograph by infiltration, surface retention and transient storage, so that at the inlet one observes a much modified inlet hydrograph, a temporal description of inlet flow (lower left insert, Figure 2). In addition, the combined flow and quality processes produce an inlet pollutograph, a time-concentration graph of a particular pollutant as it leaves the Surface Runoff Subsystem and enters the waste water conveyance system (lower right insert, Figure 2). These two graphs, one of flow and the other of quality, comprise the output of the Surface Runoff Subsystem and are input to the Transport Subsystem.

TRANSPORT SUBSYSTEM

The Transport Subsystem is comprised of the physical works for conveying storm waters and their associated pollutant loads from all of the inlets in the system through a network of underground conduits to a point (or points) of disposal. Enroute, flow and quality are both modified by accretions to the system from other tributary areas and/or point sources of pollution. In addition, flows and pollutant concentrations are attenuated in passing through the system, the degree of modification depending on such factors as system storage, "off channel" storage, phase relationships of inflow hyetographs and pollutographs and certain hydraulic properties of the system. The two lower inserts in Figure 3 illustrate a typical set of outputs from the Transport Subsystem, a hydrograph and a pollutograph, that in turn become inputs to the receiving water.

RECEIVING WATER SUBSYSTEM

The Receiving Water Subsystem may be a stream, a lake, an estuary or a coast. Discharge into an estuary will be used for illustration.

The impact of the discharge on the estuary will probably be assessed in terms of the concentration of a particular quality constituent: its distribution in space, its persistence in time, and its frequency of exceedance of a certain critical level. For a given hydrological event, the system may be observed synoptically (at the same instant in time) or temporally (at the same point in space).* One gives the distribution in space, the other the persistence in time. From the standpoint of quality management both viewpoints are usually required for each hydrological event. To obtain frequency of exceedance of a critical level the impact on the receiving water must be observed a "statistically significant" number of times. Figure 4 illustrates typical responses ("impacts") for an estuary.

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*These are sometimes referred to as the Lagrangian and Eulerian viewpoints, respectively.

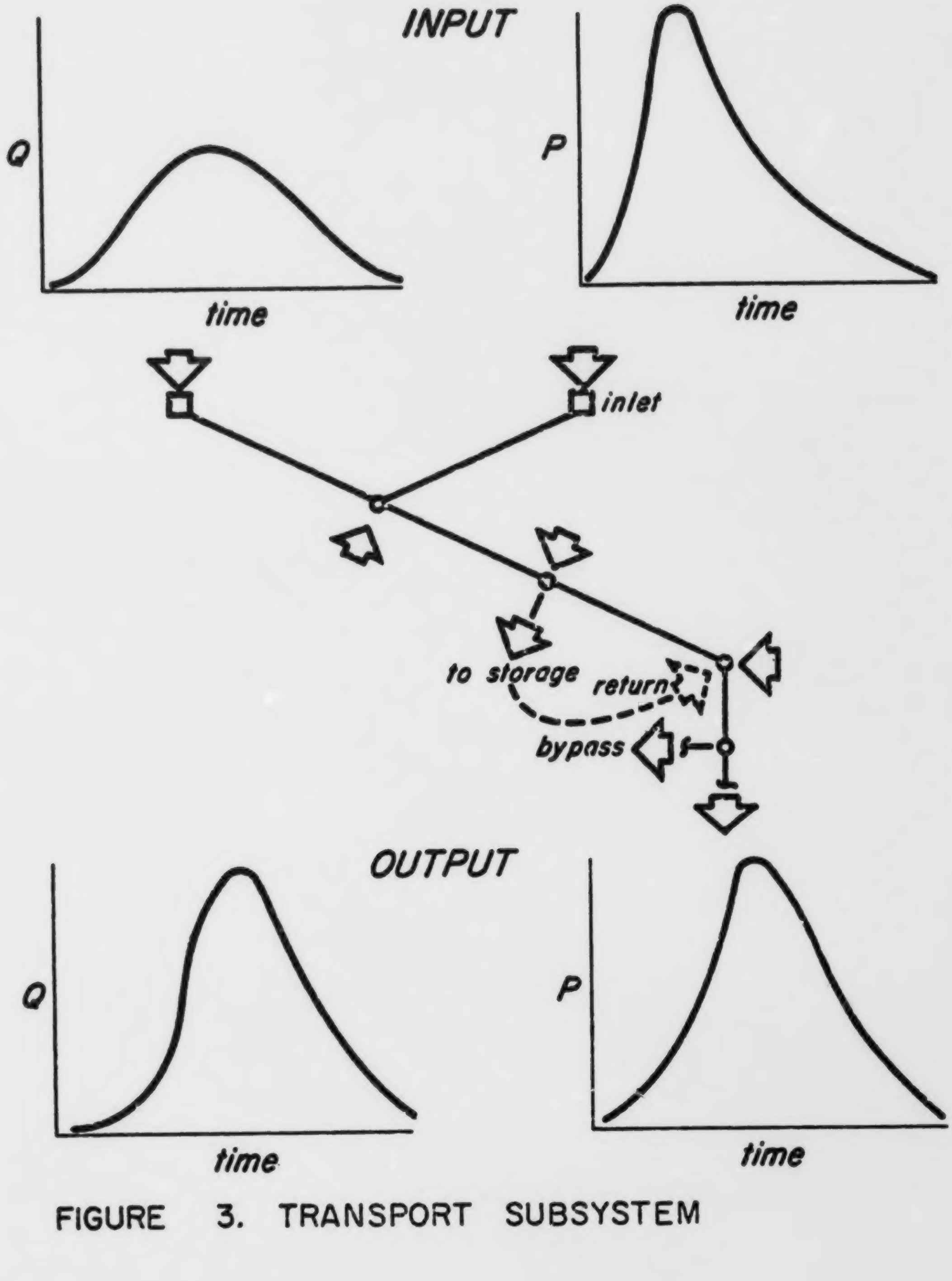

INPUT
Q
time
P
time
inlet
to storage
return
bypass
OUTPUT
Q
time
P
time

FIGURE 3. TRANSPORT SUBSYSTEM

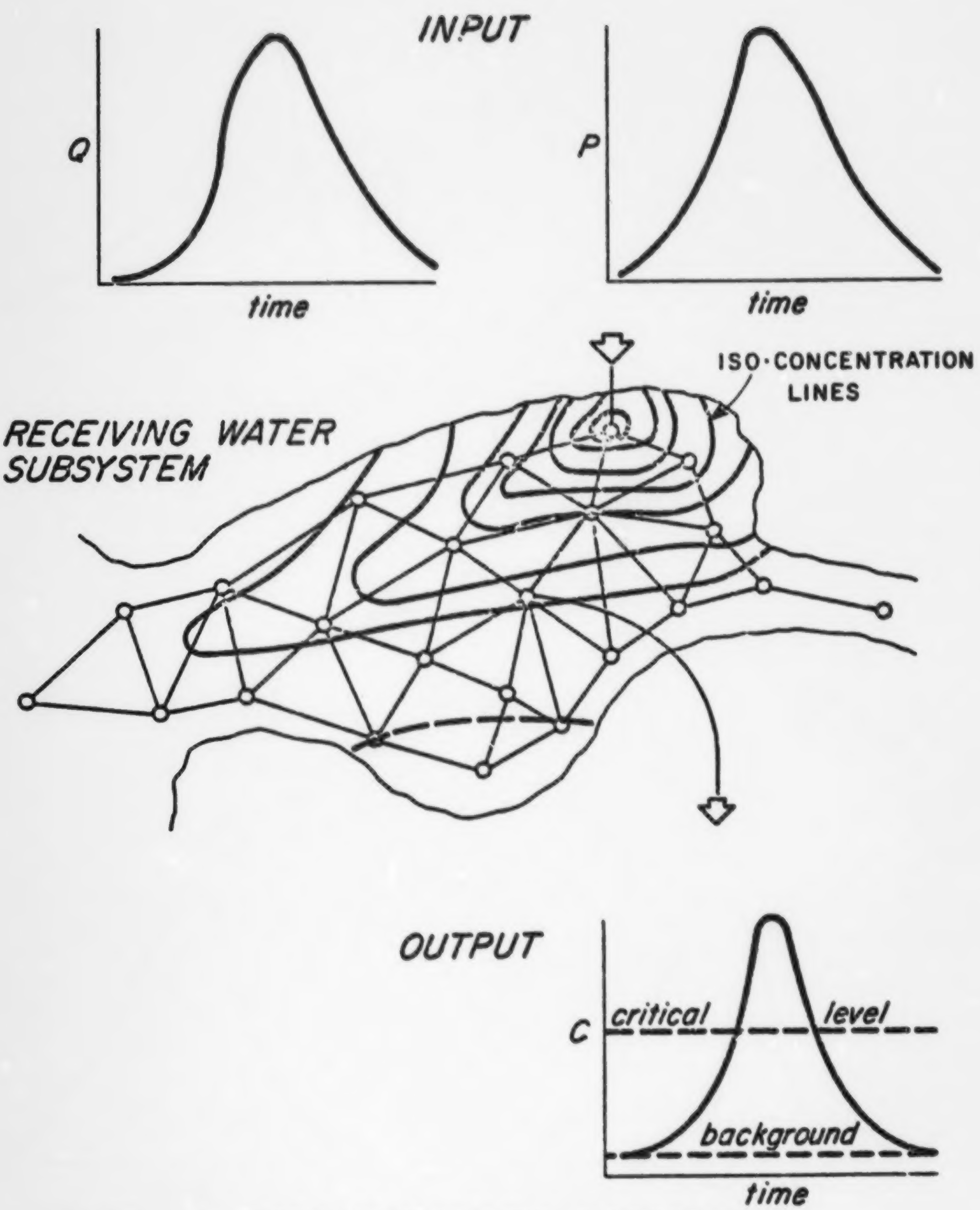

INPUT
Q
time
P
time
ISO·CONCENTRATION LINES
RECEIVING WATER SUBSYSTEM
OUTPUT
C
critical
level
background
time

FIGURE 4. RECEIVING WATER SUBSYSTEM

HYDROLOGIC EFFECTS OF URBANIZATION

The term "urbanization" refers generally to a condition in which a natural watershed has been developed for residential, commercial, and industrial purposes. The principal hydrological factor associated with urbanization is an increase in imperviousness of the watershed surface with attendant reductions in infiltration and other abstractions from storm rainfall. A summary of potential hydrological effects of urbanization is presented in Table 1. In addition to major changes in hydrology, there is evidence indicating that certain changes in the micro-climate of an urban area can be expected, Table 2.

While urbanization always increases the imperviousness of a natural watershed, the hydrological significance of this change is not always clearly evident in contrast to the postulated effects in Table 3. For example, increases in total runoff and peak flow may be offset by the use of building practices which provide for detention near the site where it occurs of rainfall excess on flat roofs or in small depressions or ponds. Another factor to be considered is the manner by which collected rainfall is discharged from roof tops. In the San Francisco Bay region, for example, collected rainfall may be disposed of in four different ways, each having a pronounced effect on the volumes and peaks of local hydrographs: discharge through a downspout into a dry well and thence into the underlying groundwater; surface storage and eventual evaporation; discharge through a downspout to a spatter block and thence over a lawn; or discharge through a downspout and pipe drain into a street gutter. Other factors to be considered in analyzing runoff from impervious areas are: the location of an impervious area with respect to the total watershed; and the proximity and geometry of the street gutter and underground drainage system.

CHARACTERISTICS OF THE URBAN WATERSHED

There is an extreme variability in the hydrological characteristics of local urban drainage systems. Figure 5 illustrates the configuration of a 47.4-acre urban watershed located in the Northwood section of Baltimore, which has some features in common with many other urban drainage basins. It is approximately three-fifths residential and two-fifths commercial, with the latter represented primarily by a large shopping center and parking lot. Average ground slope is 3 per cent and the degree of imperviousness ranges between 0.15 and 1.00 with an average value of 0.68. As indicated in Figure 5, the Northwood watershed is composed of 32 subbasins, each of which has its own land use, infiltration and runoff characteristics. The underground drainage system of Figure 5 is shown in greater detail in Figure 6.

TABLE 1. POTENTIAL HYDROLOGIC EFFECTS OF URBANIZATION

Urbanizing Influence	Potential Hydrologic Response
Removal of trees and vegetation	Decrease in evapotranspiration and interception. Increase in stream sedimentation.
Initial construction of houses, streets and culverts	Decreased infiltration and lowered groundwater table. Increased storm flows and decreased base flows during dry periods.
Complete development of residential, commercial and industrial areas	Increased imperviousness reduces time of runoff concentration thereby increasing peak discharges and compressing the time distribution of flow. Volume of runoff and flood damage potential greatly increased.
Construction of storm drains and channel improvements	Local relief from flooding. Concentration of floodwaters may aggravate flood problems downstream.

TABLE 2. CLIMATIC EFFECTS OF URBANIZATION(1)

Climatic Variable	Ratio, City to Environs
Solar radiation (insolation) in horizontal surfaces	0.85
Ultraviolet radiation, summer	0.95
Ultraviolet radiation, winter	0.70
Mean annual temperature greater in the city by 1 to 1.3°F	-
Annual mean relative humidity	0.94
Annual mean wind speed	0.75
Speed of extreme wind gusts	0.85
Frequency of calms	1.15
Frequency and amount of cloudiness	1.10
Frequency of fog, summer	1.30
Frequency of fog, winter	2.00
Total annual precipitation	1.10
Days with less than 2/10-inch of precipitation	1.10

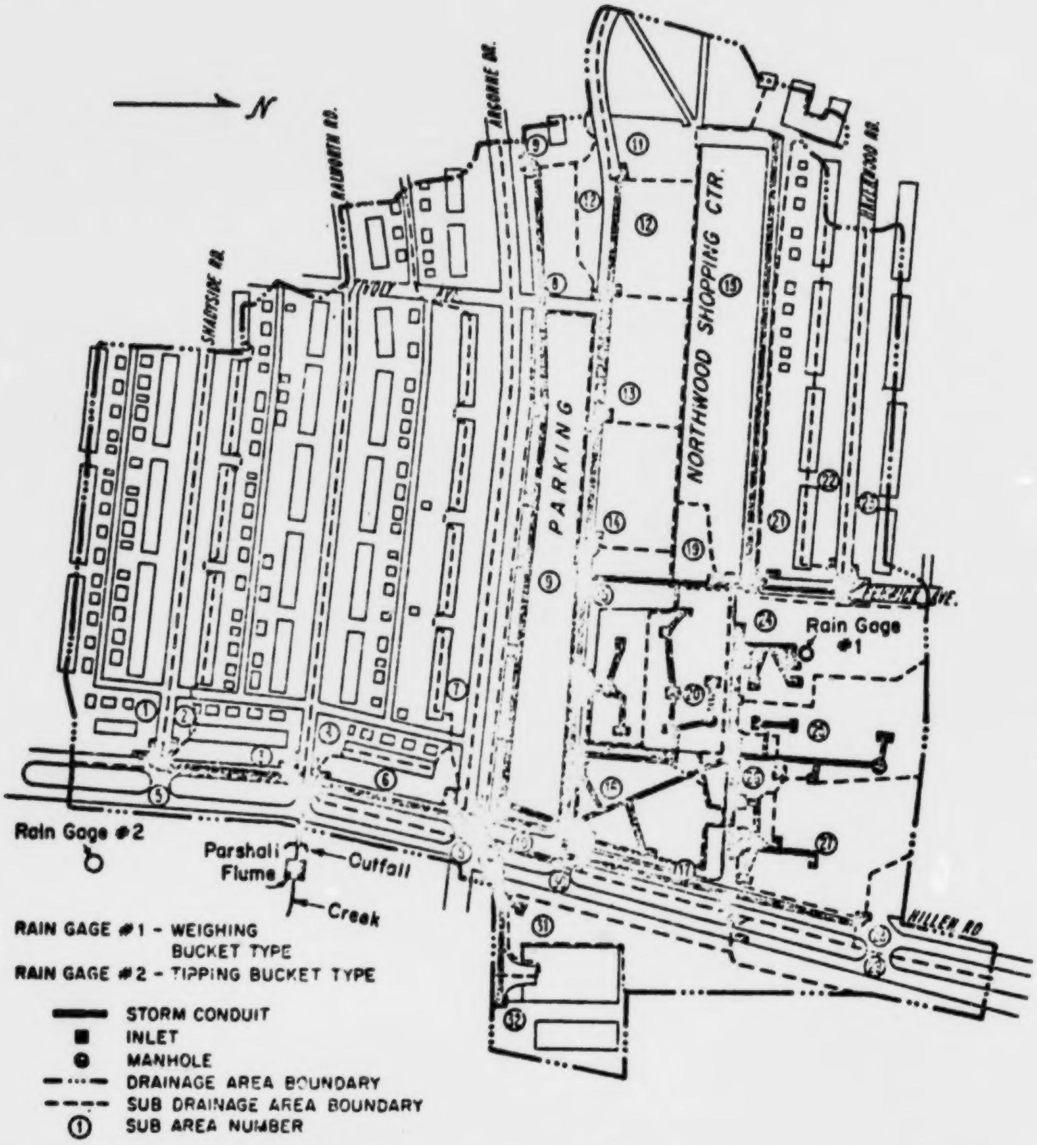

N
SHADYSIDE RD.
HALDWORTH RD.
ARGONNE DR.
TIVOLI AVE.
PARKING
NORTHWOOD SHOPPING CTR.
HILLEN RD.
Rain Gage #1
Rain Gage #2
Parshall Flume
Outfall
Creek
RAIN GAGE #1 - WEIGHING BUCKET TYPE
RAIN GAGE #2 - TIPPING BUCKET TYPE
STORM CONDUIT
INLET
MANHOLE
DRAINAGE AREA BOUNDARY
SUB DRAINAGE AREA BOUNDARY
① SUB AREA NUMBER

FIGURE 5 LAYOUT OF NORTHWOOD DRAINAGE BASIN(2)

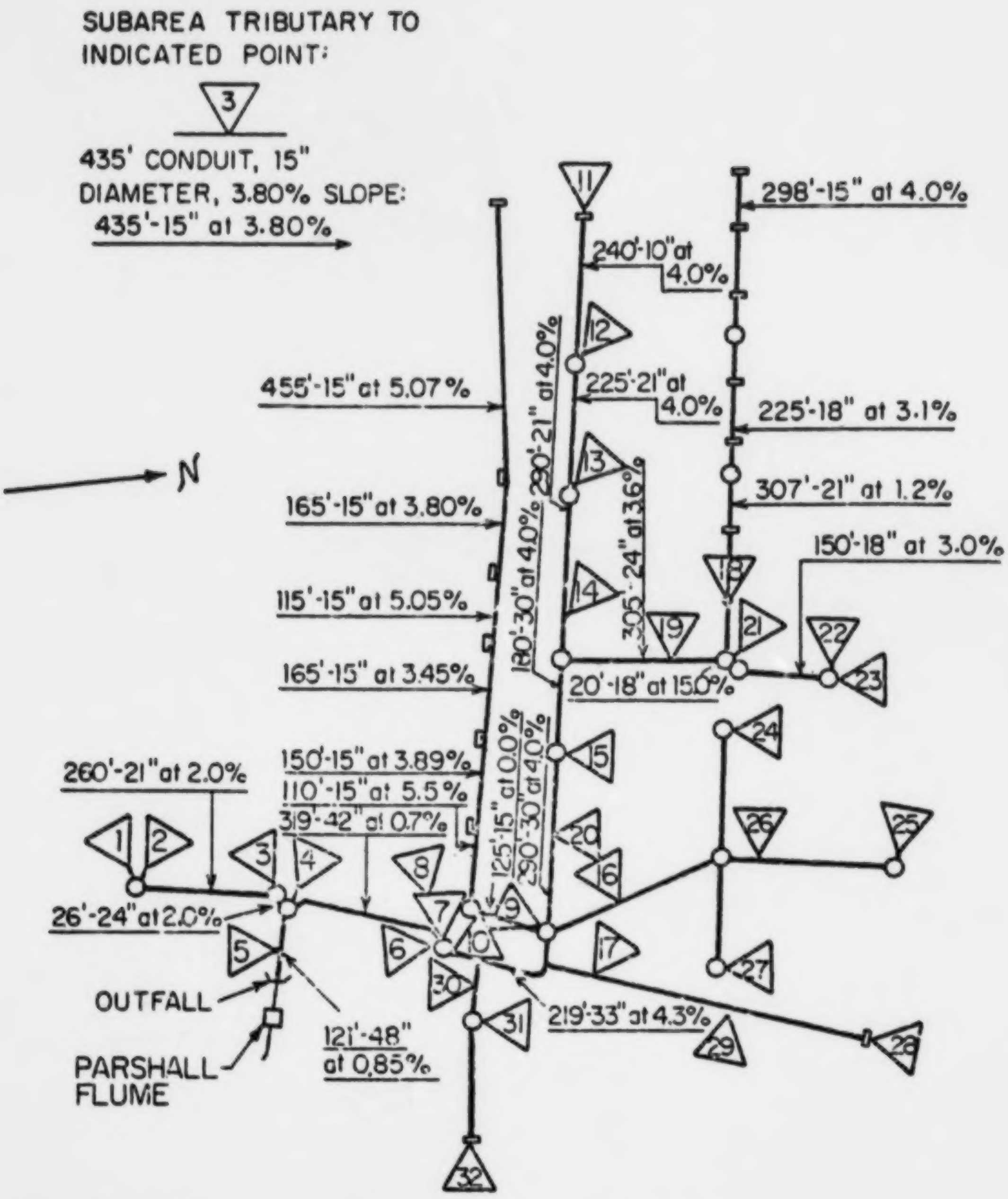

SUBAREA TRIBUTARY TO INDICATED POINT:
3
435' CONDUIT, 15" DIAMETER, 3.80% SLOPE:
435'-15" at 3.80%
N
11
298'-15" at 4.0%
240'-10" at 4.0%
12
225'-21" at 4.0%
230'-21" at 4.0%
455'-15" at 5.07%
225'-18" at 3.1%
13
307'-21" at 1.2%
165'-15" at 3.80%
305'-24" at 3.6%
18
150'-18" at 3.0%
14
180'-30" at 4.0%
115'-15" at 5.05%
19
21
22
23
165'-15" at 3.45%
20'-18" at 15.0%
24
15
150'-15" at 3.89%
260'-21" at 2.0%
110'-15" at 5.5%
125'-15" at 0.0%
290'-30" at 4.0%
319'-42" at 0.7%
26
25
20
1
2
3
4
8
16
7
9
26'-24" at 2.0%
10
6
17
5
27
30
OUTFALL
31
219'-33" at 4.3%
28
29
121'-48" at 0.85%
PARSHALL FLUME
32

FIGURE 6 SCHEMATIC DRAWING OF DRAINAGE CONDUITS.(2)

A representative rainfall hyetograph and runoff hydrograph for the Northwood watershed is shown in Figure 7. Figure 7 is notable in at least two respects. First, it illustrates the incidence of a rapid response of urban runoff to storm rainfall after initial losses are satisfied. Second, it demonstrates that losses from rainfall can be substantial when total rainfall is compared with total runoff. These losses are regulated by the particular land-use characteristics that prevailed in each Northwood subbasin at the time of the given storm.

FACTORS CONTROLLING URBAN RUNOFF

The dominant hydrologic factors in the rainfall-runoff process are rainfall, infiltration, depression storage, surface detention and gutter detention, and storage in house drains, catch basins and major sewer elements. Losses due to evaporation and to interception by vegetation are normally considered small during a storm period and can be neglected. The influence of infiltration, depression storage and surface detention, and the length of the flow plane of overland flow from a pervious area is shown conceptually in Figure 8. Figure 8 illustrates the manner in which rainfall excess can be computed by deducting time-varying losses due to infiltration, depression storage and surface detention. The overland flow hydrographs shown in Figure 8 were developed using the Izzard relationship for steady-state conditions:

$$D = 0.324 \left[\frac{0.0007\ i + c}{S^{\frac{1}{3}}} \right] L^{\frac{1}{3}}\ q^{\frac{1}{3}}$$

in which D is the surface detention depth in inches, S is the ground slope, L is the length of overland flow in feet, q is the overland flow supply in inches per hour, i is the intensity of rainfall in inches per hour and c is the coefficient of roughness. Overland flow leaving the watershed surface would next be modified by storage and translation in the street gutter and in the street lateral before it entered as an inflow to the main storm sewer.

INFLUENCE OF LAND USE ON URBAN RUNOFF

Land use exerts a profound influence on the quantity of urban runoff through its effects on imperviousness and surface cover, which in turn regulate surface depression, detention and infiltration. Representative values of imperviousness are shown in Table 3 as a function of land use classification. Representative values of depression and detention storage associated with various land cover conditions are shown in Table 4 Both Tables 3 and 4 contain assumed representative values for the Denver, Colorado area.(4) While these values of surface depression and detention are recommended only for use in connection with the Colorado Unit Hydrograph procedure, they are in general agreement with commonly assumed values of 1/16-inch for impervious areas and 1/4-inch for pervious areas. The influence of land use on infiltration-capacity is shown in Figure 9. The influence of land use on watershed outflow is further described in connection with the Selby Street example, presented below.

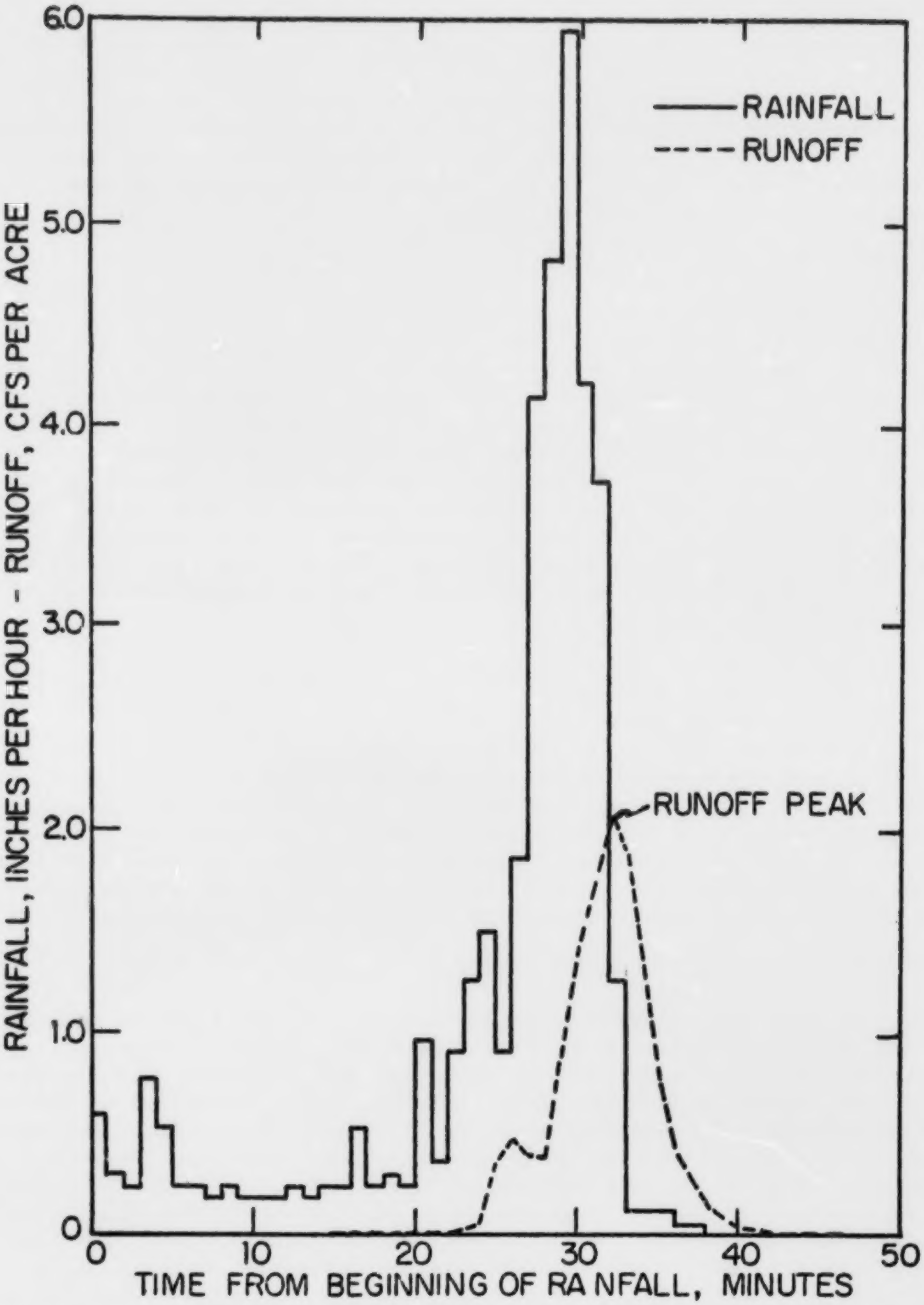


FIGURE 7 HYETOGRAPH AND HYDROGRAPH FOR STORM 8-1-65 WITH RAINFALL STARTING AT 1639-47.4 ACRE NORTHWOOD DRAINAGE AREA(2)

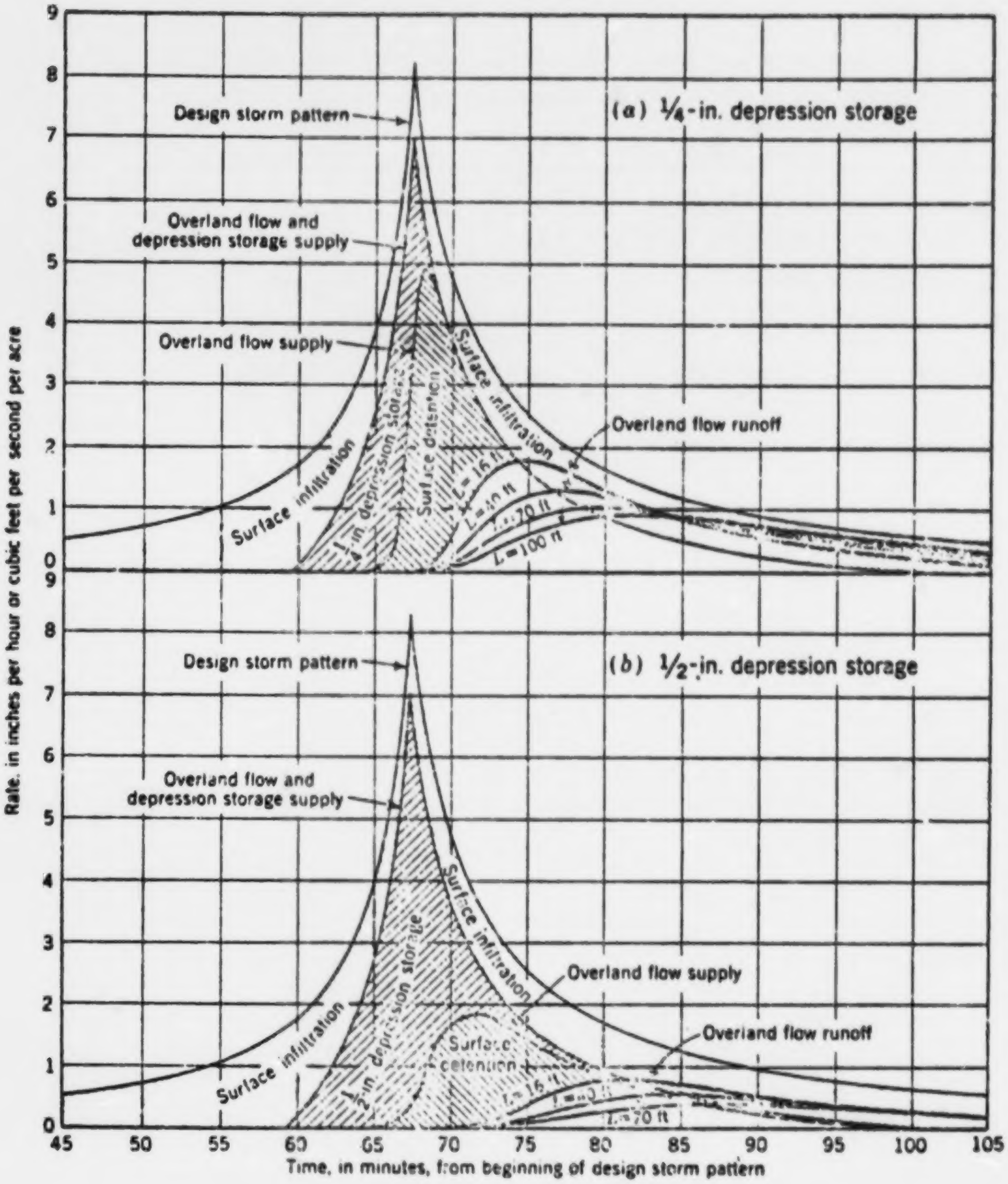


FIGURE 8 CONCEPT OF OVERLAND FLOW ON PERVIOUS AREAS[3]

TABLE 3. LAND USE VERSUS PER CENT OF IMPERVIOUSNESS[4]

Land Use	Per Cent Pervious	Per Cent Impervious
Downtown Business Area, Airport Terminal, Shopping Centers, etc.	0 - 5	95 - 100
Residential, Dense	40 - 55	45 - 60
Residential, Normal	55 - 65	35 - 45
Residential, Large Lots	60 - 80	20 - 40
Parks, Greenbelts, etc.	90 - 100	0 - 10

TABLE 4. TYPICAL DEPRESSION AND DETENTION FOR VARIOUS LAND COVERS[4]

Land Cover	Depression & Detention, Inches	Recommended, Inches
Impervious:		
Large Paved Areas	0.05 - 0.15	0.1
Roofs, Flat	0.1 - 0.3	0.1
Roofs, Sloped	0.05 - 0.1	0.05
Pervious:		
Lawn Grass	0.2 - 0.5	0.3
Wooded Area and Open Fields	0.2 - 0.6	0.4

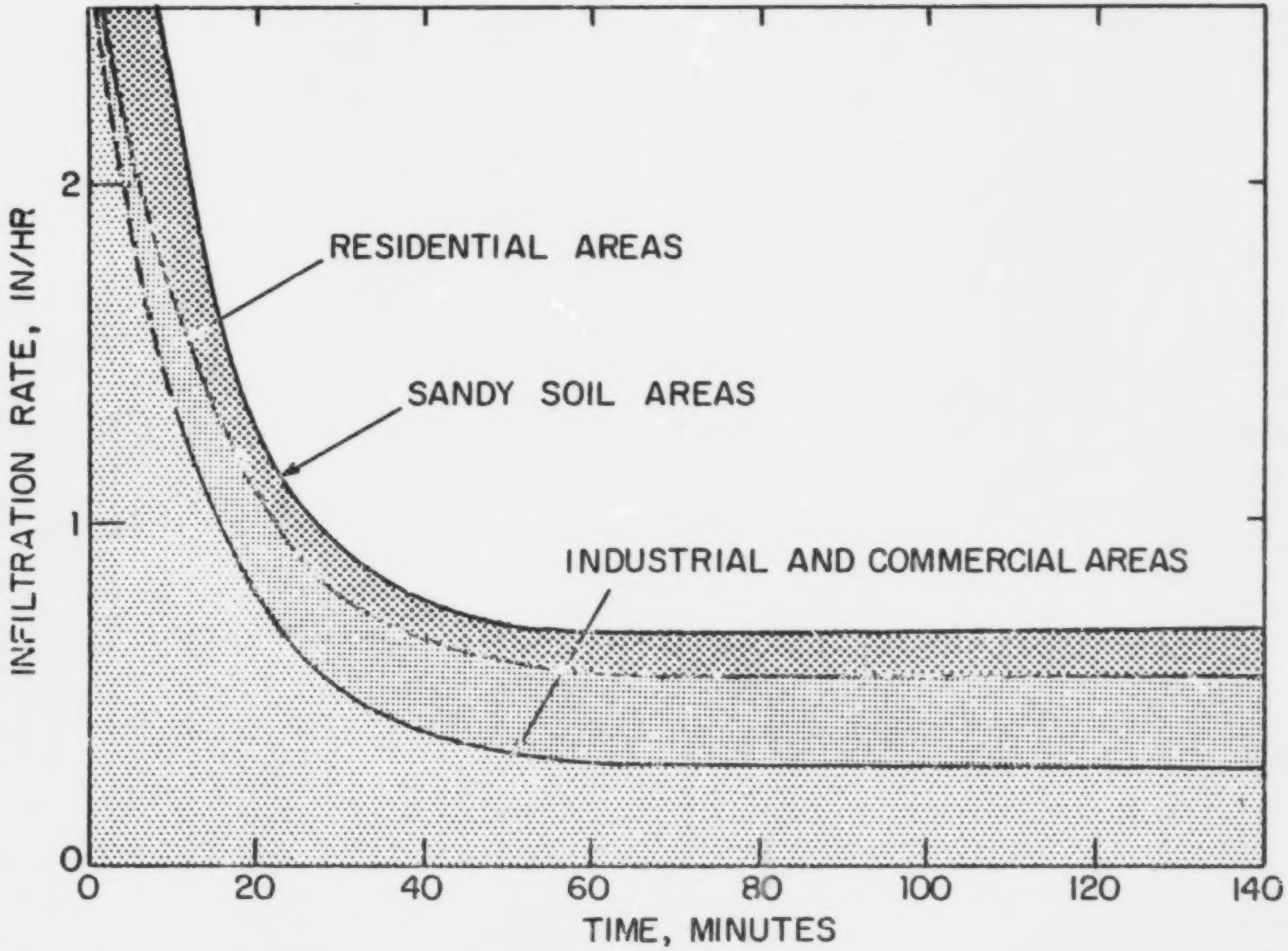


FIGURE 9 PERVIOUS SURFACE INFILTRATION-CAPACITY CURVES.(3)

CONCEPT OF A STORM DRAINAGE MODEL

Because of the complexities introduced by variable land use and hydrological conditions, storm discharge from an urban watershed can best be analyzed by dividing the total basin into smaller homogeneous units for which their individual runoff contributions can be computed. Collection of the individual subbasin outflows and their routing through main storm sewers permits determination of the total watershed outflow at the basin outlet. The concept of this approach is illustrated in Figures 10, 11 and 12.

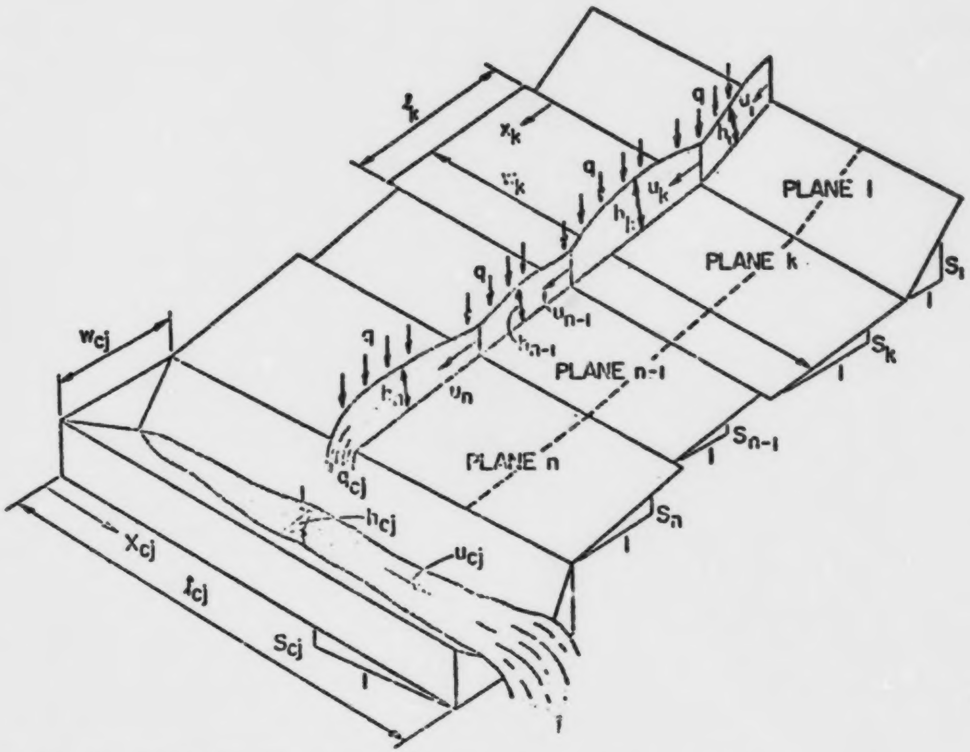
APPLICATION OF STORMWATER MODEL TO LAND-USE PLANNING

The complete stormwater model presented in Figure 3-8 has been applied to the Selby Street watershed in San Francisco in order to determine the influence on stormwater quality and quantity that might be expected with a change in land use in the Selby Street combined sewer system. The particular land use change investigated was the hypothetical conversion of a park to a multiple-family residential area. The hydrological significance of this hypothetical change would be to increase the imperviousness of the park catchment from 11 per cent to 75 per cent and to generate a new dry-weather wastewater flow from the area. The location of the Selby Street drainage area and some of the results of the computer simulation are presented in Figures 13, 14 and 15.

PONDING AND STORAGE AS A MEANS FOR CONTROLLING URBAN RUNOFF

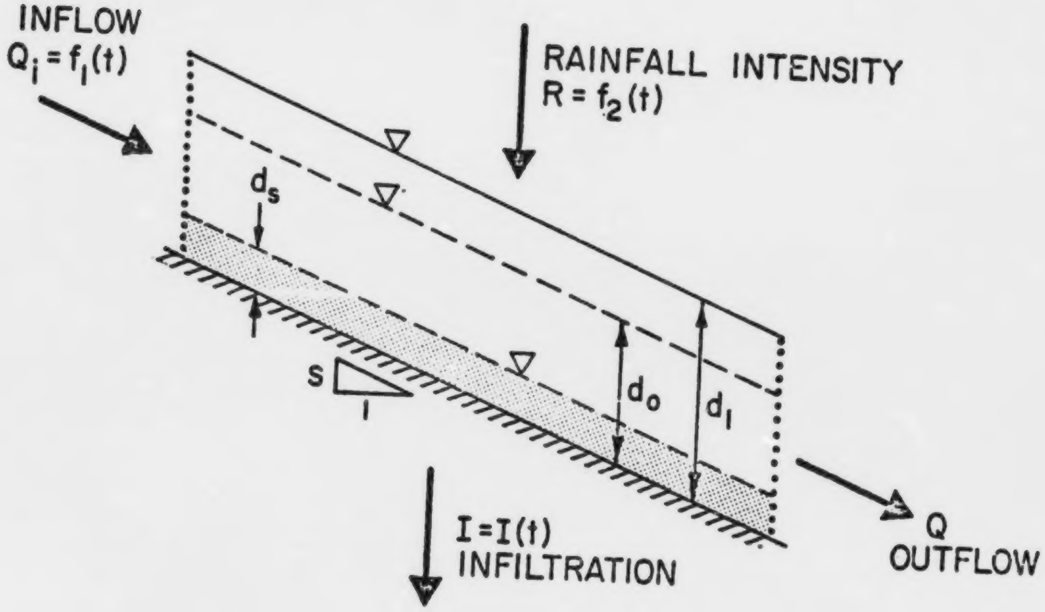
Urban runoff management problems in the Selby Street watershed center on the inability of the local treatment plant to handle combined wastewater-stormwater flows, thereby contributing to pollution in San Francisco Bay through combined sewer overflows. From the foregoing outline of factors controlling urban runoff, it is clear that storage reservoirs and detention ponds which increase runoff travel time in selected reaches of a watershed can have important advantages in reducing peak flows during stormwater runoff. Ponding can be incorporated effectively in urban storm drainage design through proper use of large flat-top roofs, plazas and other natural on-site depressions. Swales, natural channels and grassed waterways are becoming increasingly used as means for dispersing excess stormwater. In overflow areas normally reserved for recreational use, urban ponds and lakes also are being designed with provisions for temporary storage of storm runoff.

Other storage concepts include the use of in-line storage in existing main sewers. As a means for vastly reducing the incidence of combined sewer overflows, the City of San Francisco has developed a master plan which includes provision for underground detention reservoirs in a number of its watersheds.(6) Detention basin storage is coupled with a cross-system storage and transmission tunnel, and in order to take full advantages of all system capacities a total system automatic control capability will be developed.



ℓ_k
x_k
w_k
q
u_1
h_1
u_k
h_k
u_{n-1}
h_{n-1}
u_n
h_n
q_{cj}
h_{cj}
u_{cj}
w_{cj}
x_{cj}
ℓ_{cj}
S_{cj}
PLANE 1
PLANE k
PLANE n-1
PLANE n
S_1
S_k
S_{n-1}
S_n
1

FIGURE 10 CASCADE OF n PLANES DISCHARGING INTO THE j^{th} CHANNEL SECTION


INFLOW
$Q_i = f_1(t)$
RAINFALL INTENSITY
$R = f_2(t)$
d_s
S
1
d_o
d_1
$I = I(t)$
INFILTRATION
Q
OUTFLOW

INFILTRATION:

$$I = k_1 + (k_2 - k_1)\, e^{-k_3 t}$$

FLOW:

$$Q = \frac{1.49}{n} S^{1/2} \left(\frac{d_o + d_1}{2} - d_s\right)^{5/3}$$

STORAGE:

$$\frac{\Delta d}{\Delta t} = \left[R - I + \frac{Q_i - Q}{L} \right]$$

FIGURE 11 ELEMENTARY FLOW CALCULATIONS WATERSHED MODEL

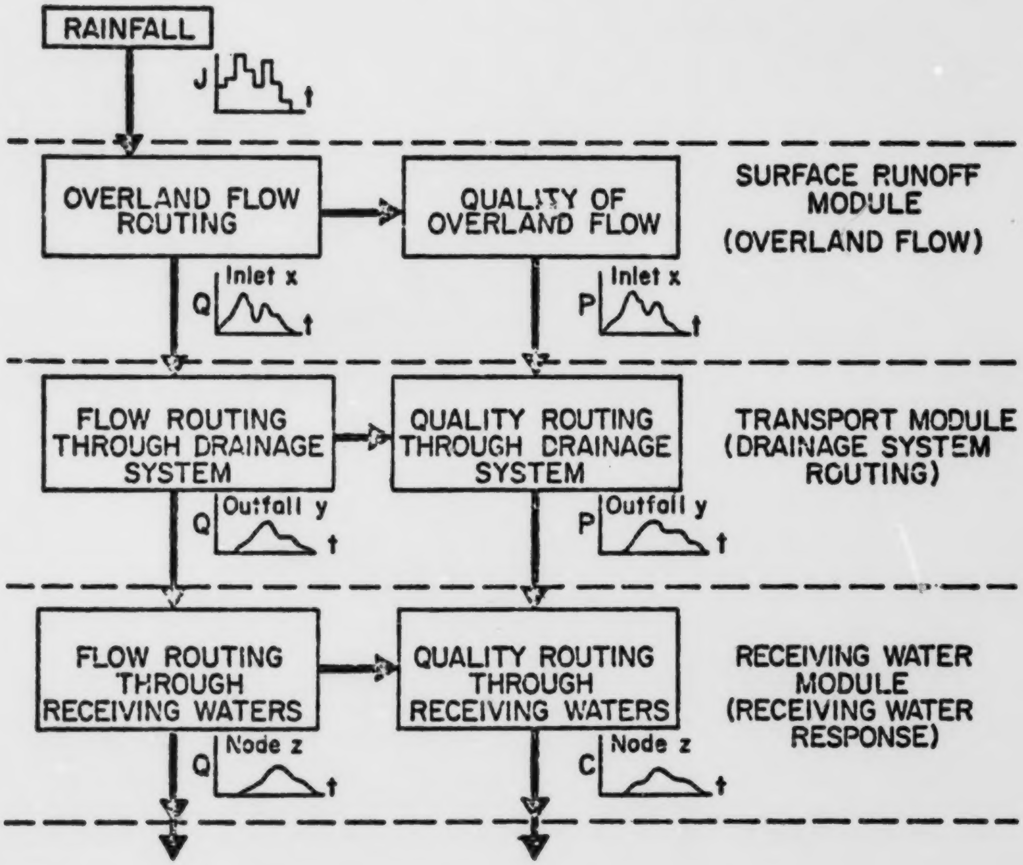

RAINFALL
J
t
OVERLAND FLOW ROUTING
QUALITY OF OVERLAND FLOW
SURFACE RUNOFF MODULE
(OVERLAND FLOW)
Q
Inlet x
t
P
Inlet x
t
FLOW ROUTING THROUGH DRAINAGE SYSTEM
QUALITY ROUTING THROUGH DRAINAGE SYSTEM
TRANSPORT MODULE
(DRAINAGE SYSTEM ROUTING)
Q
Outfall y
t
P
Outfall y
t
FLOW ROUTING THROUGH RECEIVING WATERS
QUALITY ROUTING THROUGH RECEIVING WATERS
RECEIVING WATER MODULE
(RECEIVING WATER RESPONSE)
Q
Node z
t
C
Node z
t

FIGURE 12. OVERVIEW OF THE STORM DRAINAGE MODEL

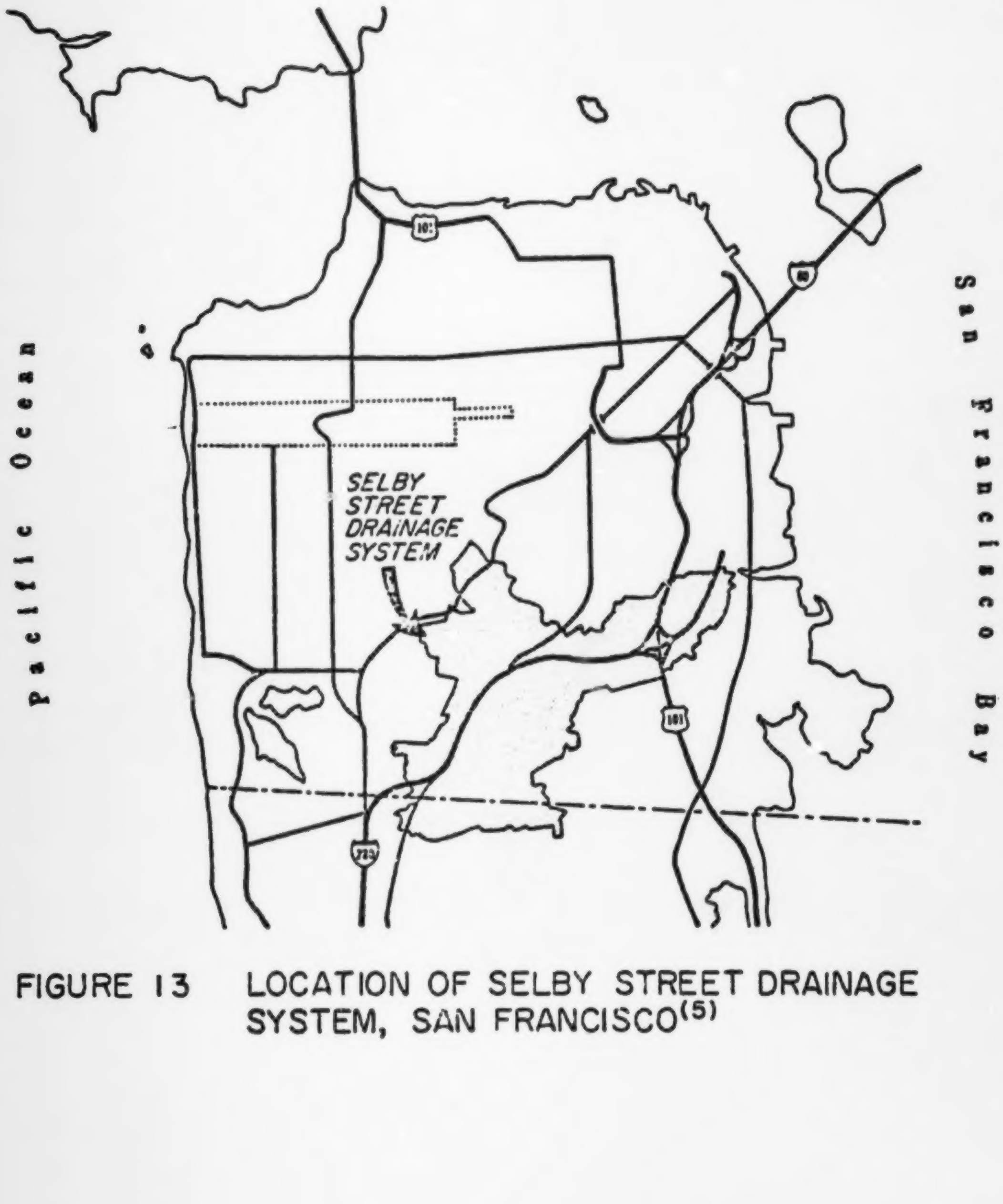

Pacific Ocean
San Francisco Bay
SELBY STREET DRAINAGE SYSTEM
101
80
101
280

FIGURE 13 LOCATION OF SELBY STREET DRAINAGE SYSTEM, SAN FRANCISCO[5]

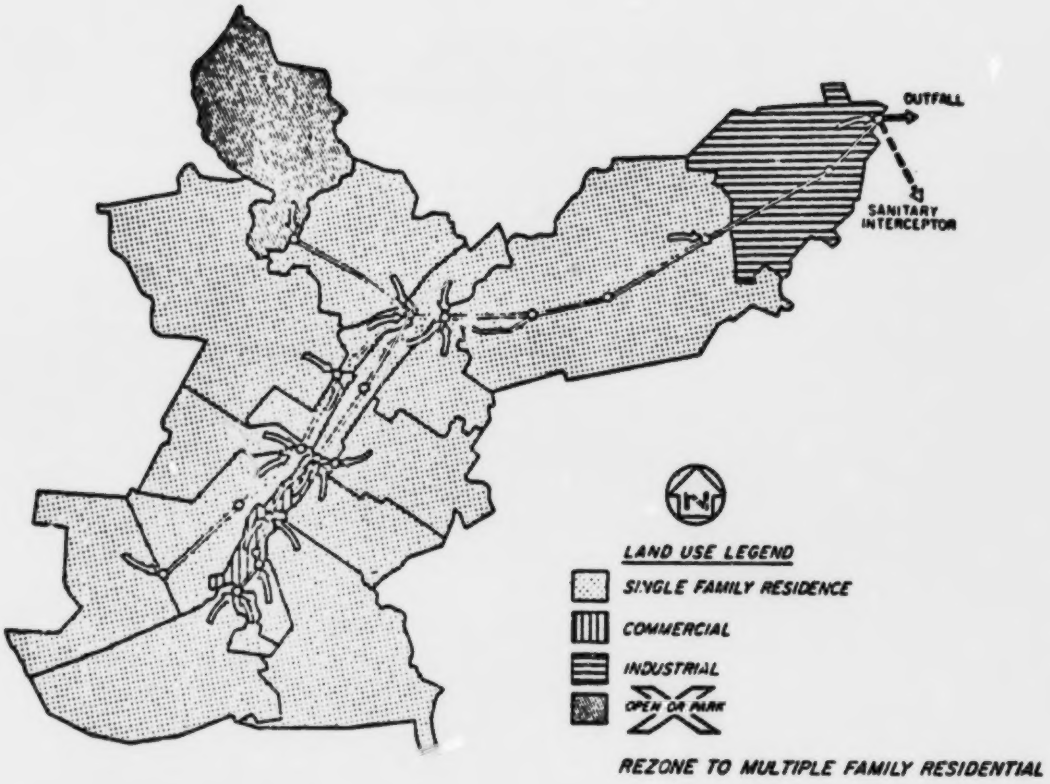

OUTFALL
SANITARY INTERCEPTOR
N
LAND USE LEGEND
SINGLE FAMILY RESIDENCE
COMMERCIAL
INDUSTRIAL
OPEN OR PARK
REZONE TO MULTIPLE FAMILY RESIDENTIAL

FIGURE 14 HYPOTHETICAL LAND USE CHANGE, SELBY STREET DRAINAGE SYSTEM(5)

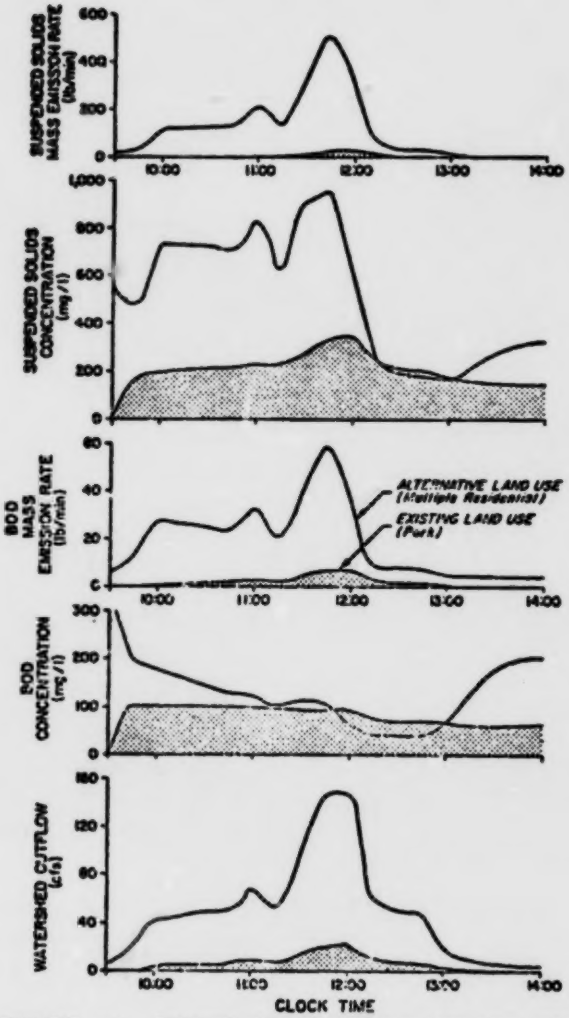


FIGURE 15 EFFECT OF CHANGED LAND USE ON CHARACTERISTICS OF SUBCATCHMENT RUNOFF, SELBY STREET (5)

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1. Lowry, William P., "The Climate of Cities," Scientific American, Vol. 217, No. 2, August, 1967.

2. Tucker, L. S., "Northwood Gaging Installation, Baltimore -- Instrumentation and Data," ASCE Urban Water Resources Research Program, Technical Memorandum No. 1, ASCE, New York, N.Y., August 1, 1968.

3. Design and Construction of Sanitary and Storm Sewers, ASCE Manuals of Engineering Practice No. 37, First Edition, 1960.

4. Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual," Denver Regional Council of Governments, Denver, Colorado, 1969.

5. Roesner, Larry A., David F. Kibler and John R. Monser, "Use of Storm Drainage Models in Urban Planning," pp. 400-405 in Proceedings of a Symposium on Watersheds in Transition, AWRA Proceedings, Series No. 14, 1972.

6. Department of Public Works, "San Francisco Master Plan for Waste Water Management, Supplement I," City and County of San Francisco, May 15, 1973.

SIMPLIFIED METHODS OF COMPUTING THE QUANTITY OF URBAN RUNOFF

By

Robert P. Shubinski*

FLOW-FREQUENCY ANALYSIS

An example of the use of statistical methods in analyzing urban runoff has been presented by Rantz(2) for the San Francisco Bay region and is discussed in part here. Regional flood-frequency relations were developed for 40 non-urban watersheds in the Bay area by means of multiple regression. The resulting regression equations are of the form $Q_T = K\ A^a\ P^b$, where Q_T is the peak discharge in cfs for a recurrence interval of T years; A is drainage area in square miles; P is mean annual precipitation in inches; K, a and b are regression coefficients. The results of the regional multiple regression analysis for the San Francisco Bay area are shown in Table 1. It is noted that other independent variables related to climatology and topography were found to have no statistical significance in the regression analysis and that drainage area and mean annual precipitation were dominant factors.

In order to illustrate the use of the flood-frequency relations in Table 1, the following hypothetical problem is posed:

Given: Drainage area = 5.0 square miles
Mean annual precipitation = 40 inches
Degree of development = 70 percent of basin urbanized and 70 percent of channels sewered or lined.

Required: Peak discharge for 25-year flood = Q_{25}.

---

*Principal Engineer, Water Resources Engineers, Springfield, Virginia.

TABLE 1. RESULTS OF FLOOD-FREQUENCY ANALYSIS-- MULTIPLE REGRESSION EQUATIONS AND ASSOCIATED STATISTICS FOR PEAK DISCHARGES AT SELECTED RECURRENCE INTERVALS(1)

Recurrence interval (years)	Multiple regression equation	Coefficient of multiple correlation	Standard error of estimate			
			Logarithmic Units	Percent		
				Plus	Minus	Mean
2	$Q_2 = 0.069\ A^{0.913}\ P^{1.965}$	0.964	0.226	68.3	40.5	54.4
5	$Q_5 = 2.00\ A^{0.925}\ P^{1.206}$	.976	.175	49.6	33.2	41.4
10	$Q_{10} = 7.38\ A^{0.922}\ P^{0.928}$	.977	.168	47.2	32.1	39.6
25	$Q_{25} = 16.5\ A^{0.912}\ P^{0.797}$	.950	.178	50.7	33.6	42.2
50	$Q_{50} = 69.6\ A^{0.847}\ P^{0.511}$	.902	.192	55.6	35.7	45.6

Solution Steps:

(1) Using the equations in Table 1 substitute drainage area and precipitation to obtain for natural watershed conditions:

Q_2 = 422 cfs

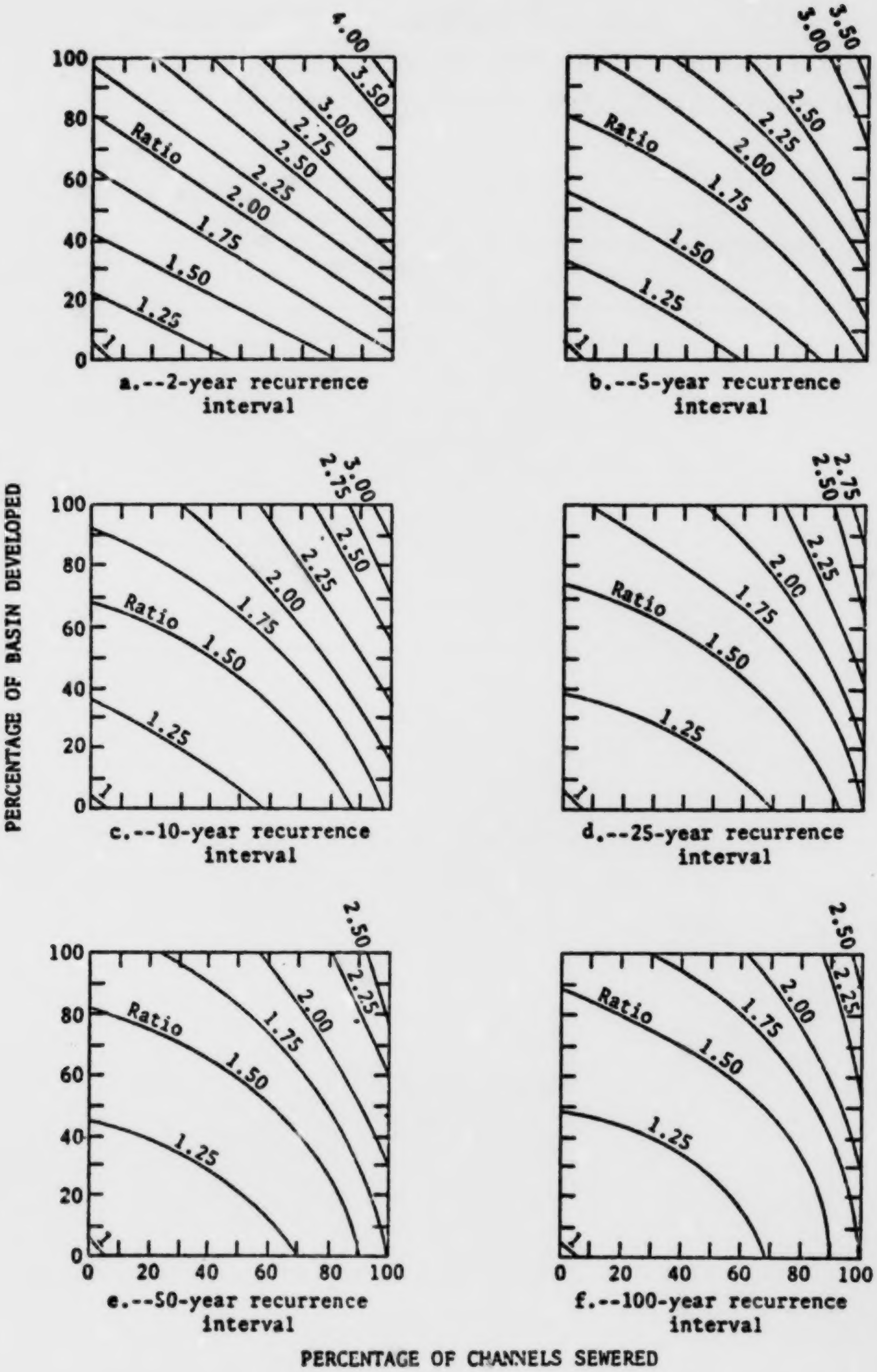
Q_5 = 758 cfs

Q_{10} = 998 cfs

Q_{25} = 1350 cfs

Q_{50} = 1800 cfs

(2) Plot the computed discharges on probability paper and fit a smooth curve.



a.--2-year recurrence interval

b.--5-year recurrence interval

c.--10-year recurrence interval

d.--25-year recurrence interval

e.--50-year recurrence interval

f.--100-year recurrence interval

FIGURE 1. RATIOS OF FLOOD-PEAK MAGNITUDE FOR URBANIZED BASINS TO THAT FOR UNURBANIZED BASINS--FOR USE WITH FLOOD-FREQUENCY METHOD$^{(1)}$.

(3) Select Q_{25} for natural conditions = 1390 cfs

(4) Adjust natural discharge for the effect of urbanization by means of the coefficient 1.90 taken from Figure 1d based on 70 percent development and 70 percent of channels sewered.

(5) Multiply the discharge of step (3) by 1.90 to obtain the required discharge = 2640 cfs.

Thus the flood-frequency method is simple to apply. One limitation lies in the fact that the regression equations of Table 1 were developed for basins larger than five square miles under essentially non-urban conditions. Increasing urbanization will tend to change the coefficients and exponents of Table 1. The flood-frequency method also is restricted to the prediction of peak flows and cannot be used in estimating the complete hydrograph.

RATIONAL METHOD

Another method of estimating urban peak flows of given frequency is the so-called rational method which is commonly applied to areas less than five square miles. Basis for the rational method is the formula:

$$Q = C\ I\ A \tag{1}$$

where Q is the maximum rate of runoff in acre-inches per hour or cfs; C is a runoff coefficient representing the integrated effects of infiltration, surface detention, and other rainfall losses; I is the average intensity of rainfall in inches per hour for a duration equal to the travel time or time of concentration for the basin; and A is the drainage area in acres. Travel time or time of concentration is the time required for flow from the most remote part of the watershed to reach the design site.

The two principal assumptions of the rational method are that: (1) the maximum rate of runoff occurs when the entire area is contributing flow; and (2) the maximum rate of rainfall occurs during the time of concentration and the design rainfall depth during the time of concentration can be converted to an average rainfall intensity. The use of the rational method is best illustrated by means of an example which has been presented previously by Rantz.

Given: Drainage area of 5.0 square miles
Mean annual precipitation = 40 inches
Degree of development is medium residential on 70 percent of the watershed and natural on 30 percent.
Overland flow path is 500 feet long at 5 percent slope in the undeveloped part of the watershed.
Average channel characteristics below undeveloped area are: length = 30,000 feet; slope = 4 percent; Manning n = 0.030; cross-section area A = 200 fq. ft. (40 ft. wide x 5 ft. deep) with wetted perimeter = 50 feet.

Required: Peak discharge for a 25-year flood for storm sewer design.

Solution Steps:

(1) Compute overland and channel travel time to get time of concentration for watershed and design rainfall duration:
   a. runoff coefficient for undeveloped area from Table 2 is C = 0.30
   b. from Figure 2 for travel path of 500 feet and C = 0.30 overland travel time = 22 minutes

TABLE 2. REGIONAL DESIGN VALUES OF PERCENTAGE OF IMPERVIOUS AREA AND OF C IN THE RATIONAL METHOD (1)

Type of development (1)	Density, in units per acre (2)	Percent impervious		C, in Rational Method	
		Santa Clara County (3)	San Francisco Bay Region (4)	ASCE (5)	San Francisco Bay Region (6)
Residential:					
Hill areas	0.5- 2	6	8	--	0.11-0.30
Low urbanization	3 - 6	10	15	0.25-0.40	.21- .38
Medium urbanization	7 -10	20	25	.30- .50	.32- .52
Heavy urbanization (apartments)	11 -20	32	40	.50- .70	.45- .70
Industrial:					
Nonmanufacturing		50	60	.60- .90	.58- .88
Manufacturing		40	50	.50- .80	.52- .79
Reserve		20	25	--	.32- .52
Commercial		50	60	.50- .95	.58- .88
Transporation		70	75	.70- .95	.60- .90
Public buildings		40	50	--	.52- .79
Public parks		12	12	.10- .25	.16- .32
Agricultural		4	4	--	.10- .30
Natural watersheds		2	2	--	.10- .30

c. substitute channel properties in Manning equation to get a channel velocity of 25.2 fps for the flood wave and a travel time = 20 minutes in the channel

d. total basin travel time = 22 + 20 = 42 minutes

(2) Compute basin-wide coefficient of runoff by weighting both the residential and undeveloped parts of the basin:

a. according to Table 2 a medium residential area has 25 percent of its area impervious and C = 0.52

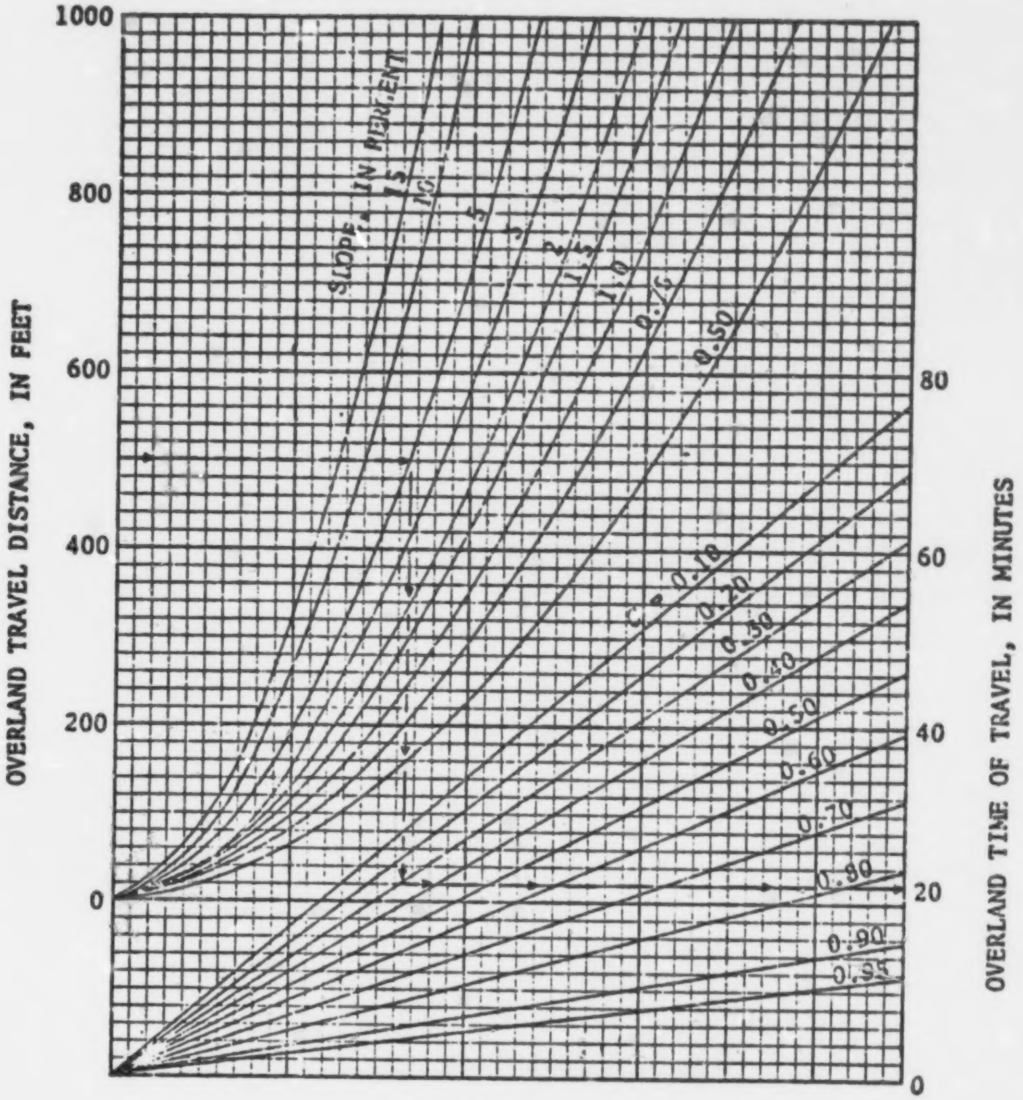


FIGURE 2. RELATION OF OVERLAND TIME OF TRAVEL TO OVERLAND TRAVEL DISTANCE, AVERAGE OVERLAND SLOPE, AND COEFFICIENT C--FOR USE IN RATIONAL METHOD (Based on Figure 3-1, Wright-McLaughlin Engineers, Ref. 2, Adapted and extended by S. E. Rantz)

b. for the undeveloped portion C = 0.30

c. weighted basin-wide C = 0.70 x .52 + 0.30 x .30
= 0.45

(3) Compute precipitation intensity, I

a. first get the rainfall depth corresponding to a recurrence interval of 25 years and a duration of 42 minutes from depth-duration-frequency data for San Francisco Bay region (TP 40 by USWB).

1. 25-year r.f. for 30 minutes = 0.88 inch and for 60 minutes = 1.12 minutes

2. by interpolation 25 year r.f. for 42 minutes = .98 inches

b. rainfall intensity = 60 x 0.98/42 = 1.40 iph

(4) Peak discharge Q_{25} = CIA = 0.45 x 1.40 x 3,200 acres
= 2020 cfs

The rational method has been used widely by drainage engineers for many years and has been applied satisfactorily to small watersheds less than five square miles in area. A basic disadvantage is the assumption of average rainfall intensity throughout the time of concentration since it is usually the intense, short-duration intensities which normally produce major runoff events. The assumption of a gross runoff coefficient is also a short-coming, although it can be offset by dividing the total watershed into smaller units and summing the individual contributions from each to get total peak discharge. Again no routing is performed and the total hydrograph is not constructed in the normal application of this method.

UNIT HYDROGRAPH METHOD

The unit hydrograph procedure is commonly employed in larger drainage and flood control studies where the total hydrograph is required in the design analysis. Briefly, the unit hydrograph (UH) is defined as the hydrograph produced by one inch of rainfall excess occurring uniformly over the basin area at a uniform rate during a specified duration. The UH is based upon the principal assumption that the ordinates of a direct runoff hydrograph are proportional to the ordinate of the UH times the rainfall excess in a given time interval. The UH procedure thus represents a linear, time-invariant process.

Use of the UH procedure involves two basic steps: (1) derivation of the UH for specified duration; and (2) application to a given rainfall event to determine the corresponding hydrograph of direct runoff. The basic steps in deriving a UH from known rainfall-runoff data are:

(1) Determine rainfall excess available to direct runoff process;
(2) Separate baseflow from corresponding hydrograph;
(3) Determine volume of direct runoff;
(4) Divide ordinates of hydrograph (without baseflow) by volume of direct runoff.

Since the UH derived from a single storm may be in error it is normally desirable to average the UH from a number of runoff events and thereby obtain an average UH of known duration for a given basin. Figures 3, 4, and 5 and Table 3 illustrate the basic steps in developing a six hour UH from a rainfall which delivered 4.63 inches of rainfall excess over a six hour period on a 40 square mile drainage basin.

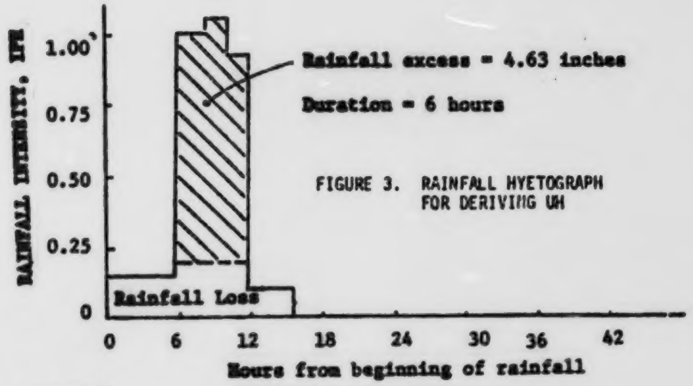


FIGURE 3. RAINFALL HYETOGRAPH FOR DERIVING UH

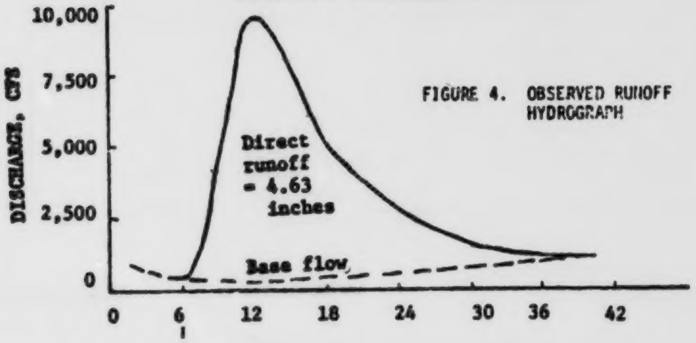


FIGURE 4. OBSERVED RUNOFF HYDROGRAPH

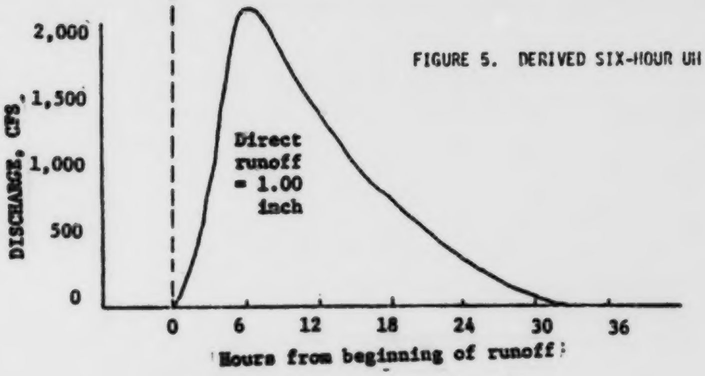


FIGURE 5. DERIVED SIX-HOUR UH

TABLE 3. STEPS IN COMPUTING SIX-HOUR UH FROM RAINFALL IN FIGURE 3

Day	Hour	Total Flow	Base Flow	Direct Runoff	Unit Hydrogr. Ordinates	Hours After Start
	hr	cfs	cfs	cfs	cfs	
1	6	500	500	0	0	0
	8	5,600	450	5,150	1,120	2
	10	9,200	400	8,800	1,915	4
	12	10,100	400	9,700	2,110	6
	14	7,800	450	7,350	1,600	8
	16	6,600	450	6,150	1,340	10
	18	5,550	500	5,050	1,100	12
	20	4,700	550	4,150	900	14
	22	4,000	600	3,400	740	16
	24	3,300	600	2,700	590	18
2	2	2,700	600	2,100	460	20
	4	2,300	650	1,650	360	22
	6	1,950	650	1,300	280	24
	8	1,650	700	950	210	26
	10	1,400	700	700	150	28
	12	1,200	750	450	100	30
	14	1,000	750	250	50	32
	16	800	800	0	0	34
Total				59,850	= 119,700 cfs-hours	

From the data in Table 3 the area under the observed hydrograph of Figure 4 represents 4.63 inches of runoff as follows:

$$\text{Runoff Volume} = \frac{119{,}700 \text{ cfs-hours} \times 3600 \, \frac{\text{sec}}{\text{hour}} \times 12 \, \frac{\text{inches}}{\text{feet}}}{40 \text{ square miles} \times \left(5280 \, \frac{\text{feet}}{\text{mile}}\right)^2}$$

$$= 4.63 \text{ inches} = \text{depth of rainfall excess.}$$

It is noted here that the six-hour UH of Figure 5 can easily be converted to UH of either shorter or larger duration by techniques of superposition or addition of UH lagged by appropriate time intervals.

To demonstrate the use of the six-hour UH derived in Figure 5, we now want to synthesize a hydrograph of streamflow resulting from a pattern of rainfall excess shown in Figure 6. The computation of rainfall excess is based upon known rainfall rates and estimates of infiltration and other losses. The increments of rainfall excess for successive six-hour periods are then multiplied by the UH ordinates of Figure 5 and the resulting runoff rates are summed as shown in Table 4. An arbitrary baseflow has been added to obtain a total basin hydrograph as shown in Figure 7.

TABLE 4. DETERMINATION OF DIRECT RUNOFF HYDROGRAPH FROM RAINFALL PATTERN IN FIGURE 6

Day	Hour	0.3 x UH ord.	0.2 x UH ord.	0.1 x UH ord.	Base Flow-cfs	Total Flow-cfs
1	0000	0			100	100
	0600	630	0		150	780
	1200	330	420		150	900
	1800	180	220	0	200	600
	2400	80	120	210	150	560
2	0600	30	50	110	100	290
	1200	0	20	60	100	180
	1800		0	30	100	130
	2400			10	50	60
	0600			0	20	20

A current example of the application of the UH is in San Diego County where the UH is used to estimate the inlet hydrograph for a number of developing watersheds in the San Diego region. In this application the UH is part of a comprehensive watershed simulation program to evaluate both the hydrologic effects of urbanization and the costs of required drainage facilities.

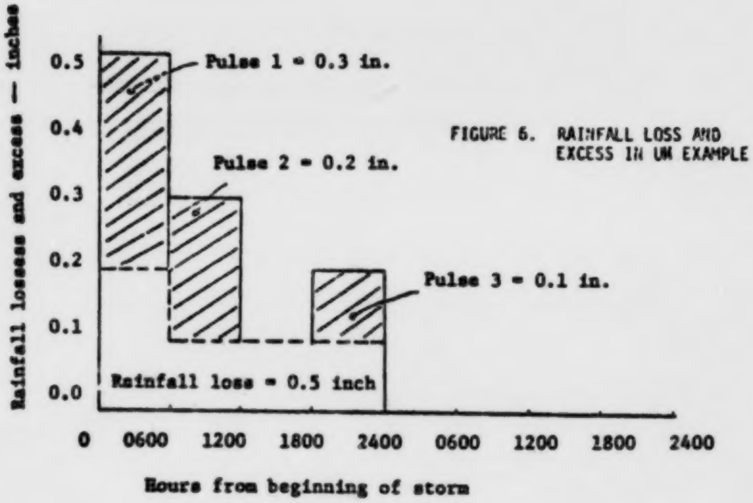


FIGURE 6. RAINFALL LOSS AND EXCESS IN UH EXAMPLE

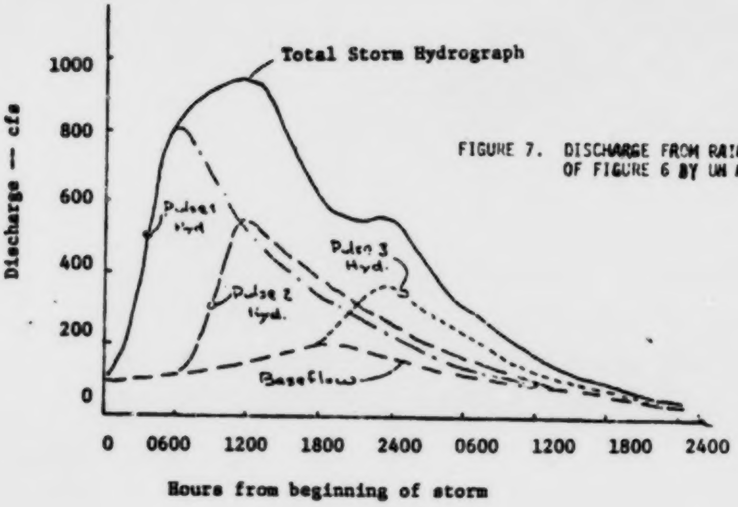


FIGURE 7. DISCHARGE FROM RAINFALL OF FIGURE 6 BY UH METHOD

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THE WRE STORM MODEL

By

Robert P. Shubinski*

SECTION 1
INTRODUCTION

BACKGROUND

It has only been within the last decade that the real pollution potential of urban runoff has come to be recognized. In a report published in 1964 by the U.S. Public Health Service [1],* the nationwide significance of pollution from urban runoff was first identified. Since that time, large amounts of effort and money have been devoted to the characterization of the quality of urban runoff and to the development of methodologies and processes to control this source of pollution. Funding for these studies has come from a number of municipalities, some states, and from federal agencies, notably the Environmental Protection Agency (EPA) and recently the U.S. Army Corps of Engineers.

A review of these studies (see References 8 and 9) shows that much work has been done in the following areas:

1. Development of stormwater treatment processes;
2. Sewer system control to maximize pipeline storage, thereby reducing the amount and frequency of overflows; and
3. Characterization of the quality of stormwater and combined sewer discharges.

In addition, several sophisticated mathematical models have been developed (some with funds from the private sector) that describe the time-varying hydraulic response of an urban drainage system to rainfall. A few of these models include descriptions of the quality of urban runoff. The EPA Stormwater Management Model [2] is a typical example of the detail and scope contained within these models.

The information and technological tools that are presently available are deficient, however, in that they do not adequately address some of the initial questions that must be answered in the preliminary planning stage. One of the questions that must be answered prior to developing a pollution control plan is: what is the present and expected future magnitude of pollution loads carried by urban runoff from a given watershed? Extensions of this question include such things as 1) what is the pollution load for an average event, 2) what is it for an extreme event, and 3) how often does a given extreme event occur?

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*Principal Engineer, Water Resources Engineers, Springfield, Virginia.

Given that we can answer these questions, it is then possible to identify some constraints for the stormwater system that is ultimately designed so that the receiving waters will be adequately protected (which is the whole idea of "stormwater management" in the first place). Therefore, it is necessary to identify those systems (i.e., combinations of treatment rate and storage volume) that can meet the constraints. The extremes of these combinations are obvious: All the runoff could be treated as it arrives at the treatment plant, or all the runoff could be stored for later treatment at a conventional treatment plant during off-peak hours. Either of these two alternatives, however, will normally prove to be highly uneconomical. In between these two extremes lie an extremely large number of treatment rates and storage capacities that will satisfy the environmental constraints placed on the system. The problem is to identify the feasible combinations.

REPORT PURPOSE

The purpose of this document is to present an analytical method that can be used in the preliminary planning stage to help answer the types of questions posed above. The method has been coded into a computer program called STORM (Storage, Treatment, Overflow, and Runoff Model). This program represents a method of analysis to estimate the quantity and quality of runoff from small, primarily urban, watersheds. Nonurban areas may also be considered. Land surface erosion for urban and nonurban areas is computed in addition to the basic water quality parameters of suspended and settleable solids, biochemical oxygen demand (BOD), total nitrogen (N), and orthophosphate (PO4). The purpose of the analysis is to aid in the selection of storage and treatment facilities to control the quantity and quality of urban stormwater runoff and land surface erosion. The model considers the interaction of eight variables:

1. precipitation,
2. air temperature for snowpack accumulation and snowmelt,
3. runoff, ...
4. pollutant accumulation on the land surface, ... related to land use
5. land surface erosion, ...
6. treatment rates,
7. storage, and
8. overflows from the storage/treatment system.

Land uses accounted for in the model include: single family residential, multiple family residential, commercial, industrial, parks, and nonurban or undeveloped areas. The program is designed for use with many years of continuous hourly precipitation records. It is a continuous simulation model but may be used for selected single events.

The City of San Francisco used this program in the preliminary planning phase of their Master Plan for Stormwater Management [3]. The Corps of Engineers is currently applying STORM in several of their Urban

Studies, and East Bay Municipal Sewerage District No. 1 (which serves seven cities on the east side of San Francisco Bay) has recently used the model in an inflow/infiltration study of their sanitary sewer system.

The program has been documented by the Hydrologic Engineering Center, Army Corps of Engineers in Davis, California, and is available to the public [4].

HARDWARE AND SOFTWARE REQUIREMENTS

This program is available for the IBM 360/50, UNIVAC 1108, and CDC 6600 or 7600 computer systems. It requires about 35,000 words of core storage and a FORTRAN IV compiler that accepts multiple ENTRY statements. Input is on the card reader and possibly a tape/disk. Output is on a 132 position line printer. One to five additional tape/disk units are required for temporary storage during the processing. The only program differences among the three computer systems are due to ENCODE/DECODE type statements and the way in which multiple output files are handled. Up to three output files are generated on tape/disk for printing at the end of the job.

SECTION 2
COMPUTATION OF RUNOFF QUANTITY

CONCEPT OF "STORM"

The quantity of urban runoff has traditionally been estimated by using a design storm through frequency-duration-intensity curves or some other statistical means based on rainfall records. Such approaches normally neglect the spacing between storms and the capacity of the urban system to deal with some types of storms better than others.

Often, through natural and artificial storage mechanisms, intense short-duration storms may be completely contained within storage so that no untreated stormwater overflows to receiving waters. Alternately, a series of closely spaced, moderately sized storms may tax the system to the point that excess water must be released untreated. Consider, for example, Figure 1 which shows the response of two different systems to the same rainfall trace. System A, which has a relatively high treatment rate and a small storage capacity, will overflow during the high intensity, short duration storm. However, it will completely contain the second storm of moderate intensity and longer duration. System B, on the other hand, which has a low treatment rate and a large storage capacity, completely contains the first storm. Notice that it would also contain the second storm if the system were analyzed independently of the antecedent storm. However, in this case the spacing of the storms is such that the system analysis must include both rainstorms as a single event to accurately describe the system's response to the rainfall trace illustrated in the figure.

A storm cannot be defined by itself, but must be defined taking into account the response characteristics of the urban stormwater system. It is for this reason that an approach was developed that would not only recognize the properties of rainfall duration and intensity, but would also consider storm spacing and the capacity of the urban stormwater system.

Figure 2 shows, pictorially, the interrelationship of the eight stormwater elements considered in this approach for estimating stormwater runoff quality and quantity. In this approach, rainfall washes dust and dirt and the associated pollutants off the watershed to the storage-treatment facilities so that as much stormwater runoff as possible can be treated prior to its release. Runoff exceeding the capacity of the treatment plant is stored for treatment later. When the storage facilities become inadequate to contain the runoff the untreated excess is wasted through overflow directly into the receiving waters.

For a given precipitation record, the quantity, quality, and number of overflows will vary as the treatment rate, storage capacity,

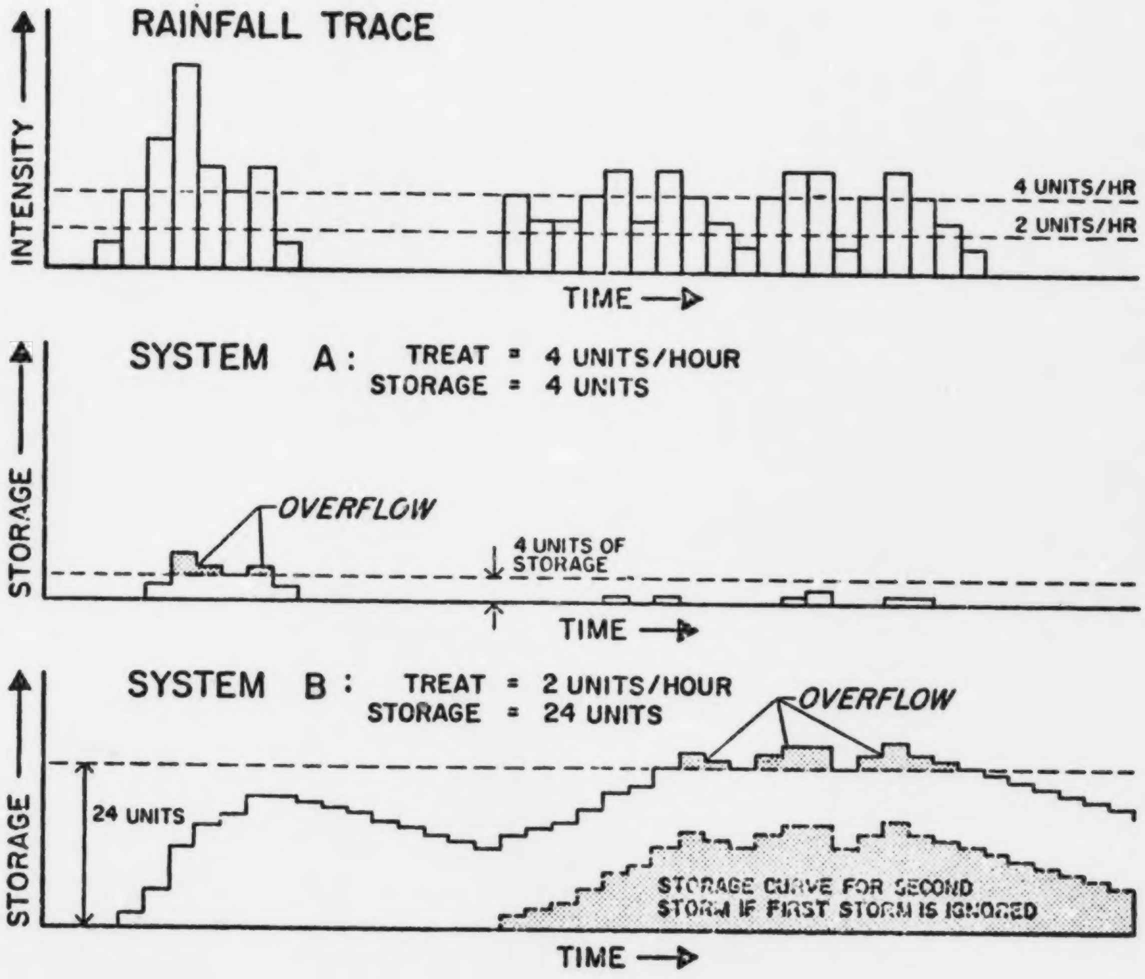

FIGURE 1
System Response Examples

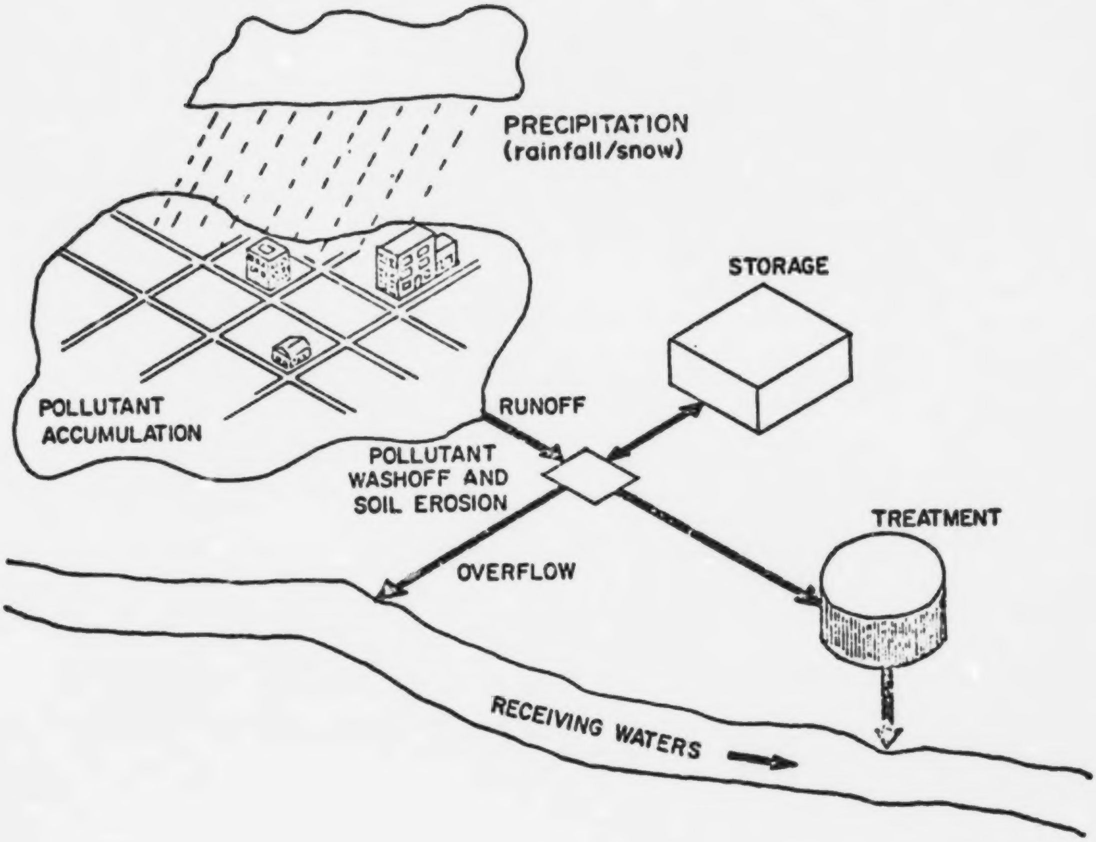
PRECIPITATION
(rainfall/snow)
STORAGE
POLLUTANT
ACCUMULATION
RUNOFF
POLLUTANT
WASHOFF AND
SOIL EROSION
TREATMENT
OVERFLOW
RECEIVING WATERS

FIGURE 2
Conceptualized View of Urban System Used in "STORM"

and land use is changed. Land surface erosion is a function of land use, soil types, ground slope, rainfall/snowmelt energy and erosion control practices. A typical method of investigation is to alter the treatment, storage, and land use and note the resulting response of the system. A group of alternatives can then be selected from among those meeting the overflow quantity and quality objectives.

COMPUTATION OF THE QUANTITY OF RUNOFF

Runoff is calculated on an hourly basis as a function of rainfall plus snowmelt using the following expression:

$$R = C(P - f) \tag{1}$$

where

R = urban area runoff in inches per hour;

C = composite runoff coefficient dependent on urban land use;

P = rainfall plus snowmelt in inches per hour over the urban area; and

f = available urban depression storage in inches per hour.

For simplicity we will omit the snowmelt computation in our discussion here. The interested reader is referred to the User's Manual [4] for details of that computation.

The runoff coefficient represents losses due to infiltration. It is computed from land use data as follows:

$$C = C_p + (C_I - C_p) \sum_{i=1}^{L} X_i F_i \tag{2}$$

where

C_p = runoff coefficient for pervious surfaces;

C_I = runoff coefficient for impervious surfaces;

X_i = area in land use i as a fraction of total watershed area;

F_i = fraction of land use i that is impervious; and

L = total number of urban land uses.

Before the runoff coefficient is applied, depression storage losses must be satisfied. Depression storage represents the capacity of the watershed to retain water in ditches, depressions and on foliage. The amount of depression storage at any particular time is a function of past rainfall plus snowmelt and evapotranspiration rates. The function is computed continuously using the following expression, where f is in inches:

$$f = f_o + N_D k , \qquad \text{for} \quad f \leq D \tag{3}$$

where

f_o = available depression storage, in inches, after previous rainfall;

N_D = number of dry days since previous rainfall;

k = recession factor, in inches/day, representing the recovery (evapotranspiration) of depression storage and infiltration in inches; and

D = maximum available depression storage in inches.

Figures 3a and 3b show graphically the hourly precipitation (P), depression storage (f), precipitation excess (P-f) and the resulting runoff (R). Figures 3b and 3c show how the runoff is distributed between treatment, storage and overflow for a system with a treatment rate of 0.02 inches/hour and a storage capacity of 0.16 inches.

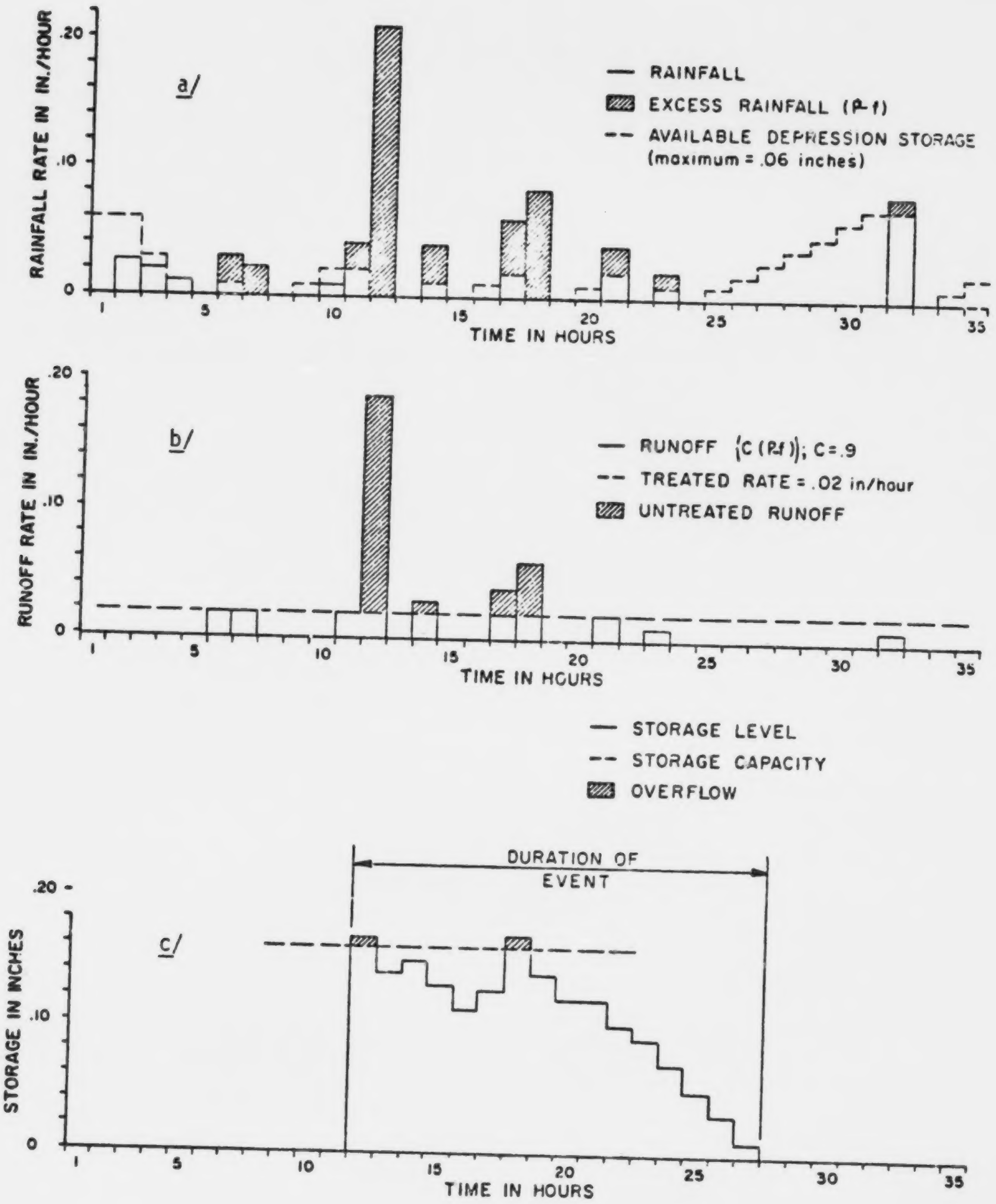

FIGURE 3
Time Histories of Rainfall, Runoff, and Storage

SECTION 3
COMPUTATION OF RUNOFF QUALITY

SOURCES OF POLLUTANTS

Basically, pollutant loads are introduced into urban runoff from three sources:

1. The land surface itself, primarily impervious surfaces;
2. Catch basins; and
3. The sewers in combined systems.

Of these three sources, the land is the most important. Catch basins can be a source of first-flush or shock pollution. An American Public Works Association (APWA) study [5] in Chicago found that:

> ". . . the liquid remaining in a basin between runoff events tends to become septic and that the solids trapped in the basin take on the general characteristics of septic or anaerobic sludge. The liquid in catch basins is displaced by fresh runoff water in the ratio of one-half the volume for every equal volume of added liquid. During even minor rainfall or thaw this displacement factor can release the major amount of the retained liquid and some solids. The catch basin liquid was found to have a BOD content of 60 ppm in a residential area. For even minor storms, the BOD of the catch basin liquid would be seven-and-one-half (7-1/2) times that of the runoff which had been in contact with street litter. Improved design of catch basins, and better operational and maintenance practices, could reduce this first-flush pollutional effect."

In combined sewer systems, wastewater is incorporated into the storm runoff. In addition, the storm runoff, as it passes through large sewers, scours sediment deposited by wastewater flows during preceding dry-weather periods. Figures 4 and 5 illustrate the effects of wastewater sewage and of catch basins and storm sewer scour on the quality of stormwater overflows [6].

As stated above, the most important contributor of pollutants to urban runoff is the land surface itself, primarily the streets and gutters and other impervious areas directly connected to streets or storm sewers. Pollutants accumulate on these surfaces in a variety of ways. There is, for example: debris dropped or scattered by individuals; sidewalk sweepings; debris and pollutants deposited on or washed into streets from yards and other indigenous open areas; wastes and dirt from building and demolition; fecal dropping from dogs, birds and other animals; remnants

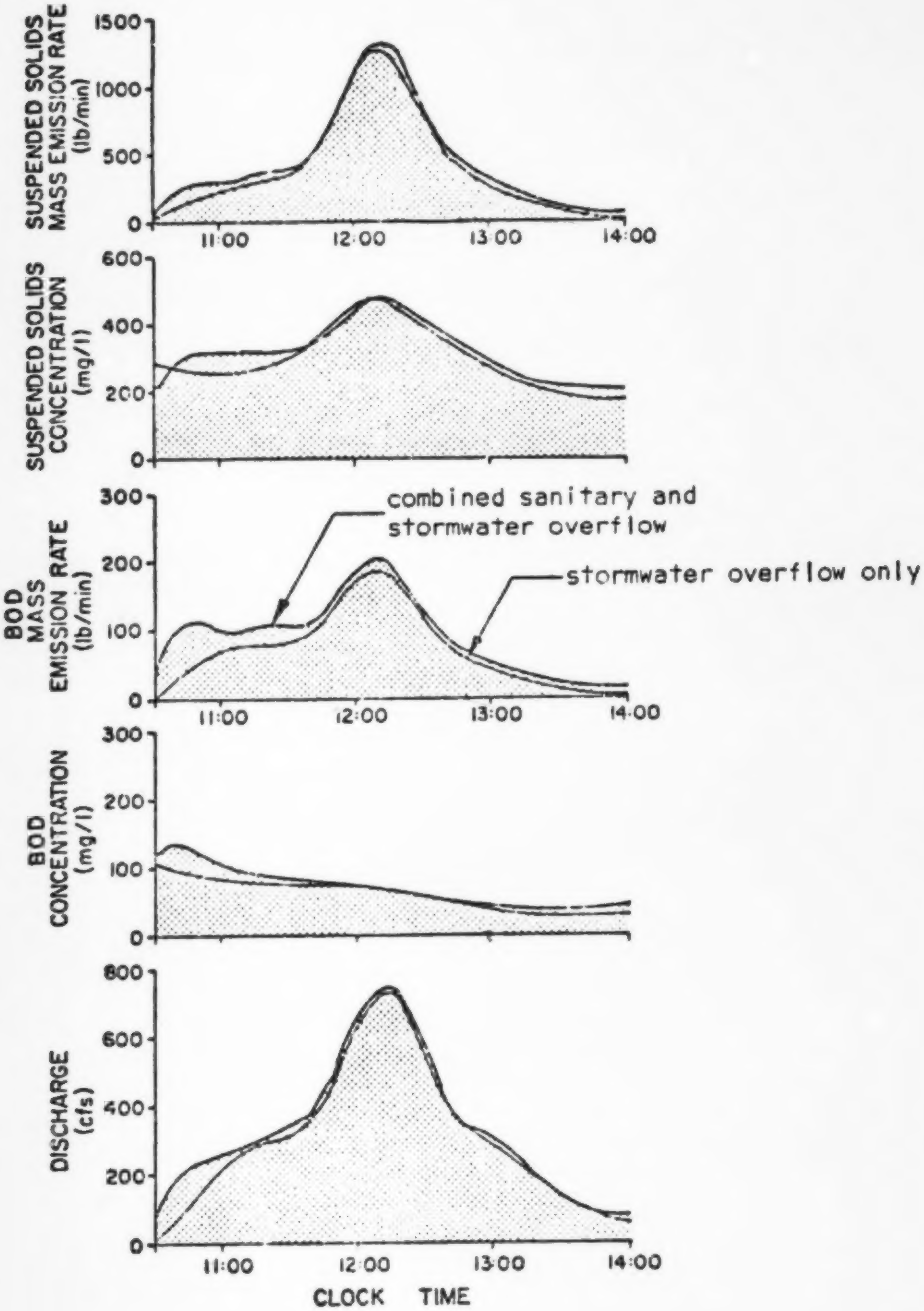

FIGURE 4
Comparison of Stormwater and Combined Overflows
(Selby Street, San Francisco)

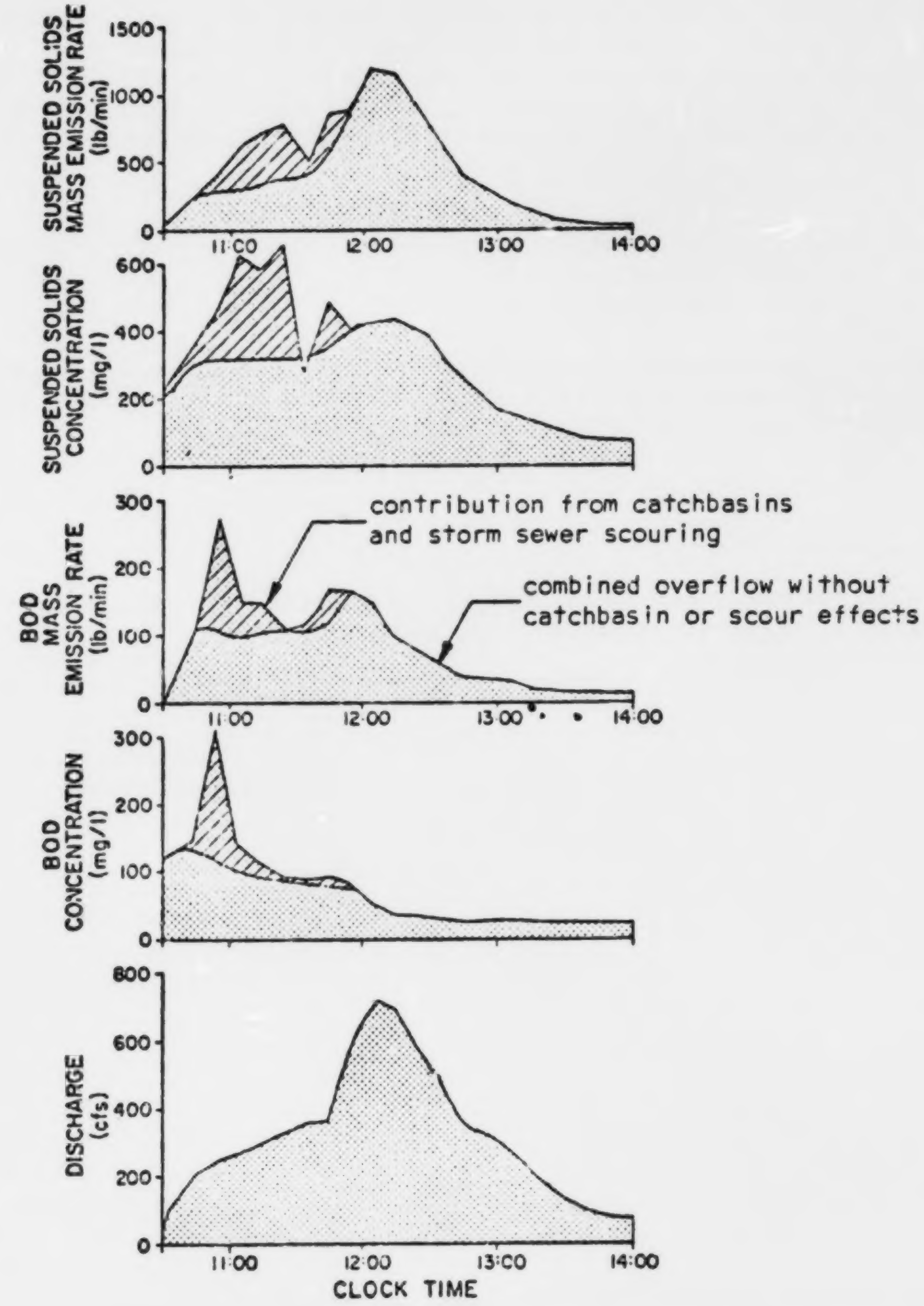

FIGURE 5
First-Flush Pollutional Effects of Catch Basins
And Sewer Scour on Combined Sewer Overflow
(Selby Street, San Francisco)

of household refuse dropped during collection or scattered by animals or wind; dirt, oil, tire and exhaust residue contributed by automobiles; and fallout of air pollution particles. The list could go on and on. Irrespective of the way in which pollutants accumulate on the urban watershed, they are generally associated with one of the following forms of street litter:

1. Rags,
2. Paper,
3. Dust and dirt,
4. Vegetation, or
5. Inorganics.

Table 1, which gives estimated street litter components for a residential area in Chicago, provides a rough measure of the relative importance of these components.

TABLE 1
Monthly Summary of Estimated Street Litter Components, From a 10-acre (4 ha) Residential Area, Chicago**

Month	Street Refuse Components (Tons/Month)					
	Rags	Paper	Dust & Dirt	Vegetation	Inorganic	Total*
Jan.	.0015	.036	.55	.00	.09	.68
Feb.	.0015	.036	.55	.00	.09	.68
March	.0015	.036	.55	.08	.09	.76
April	.0015	.036	.55	.08	.09	.76
May	.0015	.036	.55	.08	.09	.76
June	.0015	.036	.55	.08	.09	.76
July	.0015	.036	.55	.08	.09	.76
Aug.	.0015	.036	.55	.08	.09	.76
Sept.	.0015	.036	.55	.08	.09	.76
Oct.	.0015	.036	.55	.83	.09	1.56
Nov.	.0015	.036	.55	.83	.09	1.56
Dec.	.0015	.036	.55	.00	.09	.68
TOTAL*	.0180	.432	6.60	2.22	1.08	10.48

*Some totals have been rounded off.

It is readily apparent that the most significant component is dust and dirt except during the fall of the year when vegetation (primarily leaves) becomes the dominant component.

**This table is a reproduction of Table 4 in Reference 5.

TYPES OF POLLUTANTS AND LOADING RATES

Nearly all of the pollutants found in urban runoff are associated with the dust and dirt component of street litter. By type, COD, BOD, and solids (suspended and settleables) are found in the greatest quantity. Nitrogen and phosphorus are also found in significant quantities. In areas where street deicing by salting is practiced, winter runoff contains very high chloride concentrations. Other pollutants found in urban runoff include pesticides, herbicides, fertilizers and other chemical additives, heavy metals, and many other known and unknown pollutants.

Data on the rate at which pollutants accumulate on an urban watershed is very scanty. In fact, it is almost non-existent. A lot of data has been collected on the quality of combined sewer overflows and stormwater discharges for various cities in the United States (Tulsa, Oklahoma; Washington, D.C.; Atlanta, Georgia; San Francisco, California; Sacramento, California; and Roanoke, Virginia) as a result of the U.S. Environmental Protection Agency's demonstration grants program for abatement of stormwater pollution. The studies are reported in EPA's Water Pollution Research Series; however, the dissimilar forms in which the data is reported makes it difficult and in some cases impossible to generalize. The problem is that data is often presented as average concentrations or as pounds of pollutant runoff per inch of rain, and the reported values may be for combined wastewater and storm runoff rather than for storm runoff alone.

Even for a given watershed, there is no apt description of "typical" stormwater runoff characteristics because of the variablility of rainfall-runoff patterns. Thus, reports of "mean concentration" or pounds per inch of rainfall are meaningless as generalized variables and they show poor correlation with any runoff parameters of interest,

Results from a demonstration project conducted in Tulsa, Oklahoma [7], were summarized in terms of pounds of loading per day per mile of street for each of 15 areas sampled in the study. These results, presented in Table 2, give an indication of the magnitude of pollutant buildup for different land uses. These findings must be viewed with caution, however, because they were computed by taking the "average" concentration of the pollutant for all events monitored, which when multiplied by the total storm runoff gave "total annual mass emissions" converted into a rate per day per mile. A much more useful way to have developed this data would have been to sum the products of discharge and concentrations over each of the observed events and then to sum all such total storm emissions over a year.

One of the best existing sources of information on the rate of accumulation of pollutants on urban watersheds is that data collected in a field study in Chicago by APWA [5]. This study determined the rate of buildup of dust and dirt in the streets on a number of different test areas and then related the concentrations of various pollutants to the dust and dirt. A summary of their findings is contained in Table 3.

TABLE 2
Average Daily Loads Per Mile of Street*
(Tulsa, Oklahoma)

Test Area	Total Street Miles	Average Load: lbs/day/mile of street				
		BOD	COD	Total Solids	Organic Kjeldahl Nitrogen	Soluble Orthophosphate
Residential						
3	14.87	1.41	11.46	120	0.26	0.34
5	16.32	2.80	21.43	43	0.11	0.13
7	6.84	1.20	7.20	63	0.12	0.10
8	6.97	2.72	20.89	69	0.12	0.21
9	3.11	1.12	13.09	47	0.07	0.11
11	49.05	1.60	13.29	66	0.08	0.15
13	5.58	2.58	15.16	81	0.25	0.20
15	2.06	2.47	8.67	56	0.07	0.17
Commerical						
2	7.41	2.54	15.12	92	0.32	0.29
10	12.99	2.10	20.44	82	0.16	0.13
12	3.39	4.53	25.47	113	0.22	0.30
Industrial						
1	11.46	4.85	41.10	838	0.41	1.30
4	28.40	3.98	29.29	175	0.28	0.30
6	12.24	1.70	12.73	49	0.09	0.13
Average Values						
Residential		1.98	13.9	63.1	0.14	0.18
Commercial		3.06	20.3	95.7	0.23	0.24
Industrial		3.51	27.7	354.	0.26	0.57

*Reproduced from Reference 7.

TABLE 3
APWA Findings on Rate of Pollutant Buildup On Urban Watersheds*

Amount of Dust and Dirt and Strength of BOD by Land Use

Land Use	Amt. of D/D by land use lbs/day/100 ft of curb	BOD of D/D mg/g
Commercial	3.3	7.7
Industrial	4.6	3
Multiple family	2.3	3.6
Single family residence	0.7	5
Assumed weighted average	1.5	5

Amount of Pollutant by Type of Land Use

Item	Single Family	Multiple Family	Commercial
Water Soluble (mg/g)	6.0	5.6	12.4
Volatile Water Soluble (mg/g)	3.8	3.4	6.9
BOD (mg/g)	5.0	3.6	7.7
COD (mg/g)	40	40	39
PO_4 (mg/g)	.05	.05	.07
N (mg/g)	.48	.61	.41
Total plate counts/g (x 1000)	10,900	18,000	11,700
Confirmed coliform/g (x 1000)	1,300	2,700	1,700
Fecal enterococci/g	645	518	329

To convert the data contained in Table 3 into a form comparable with that of Table 2, the Dust and Dirt is multiplied by 2 (gutters per street) x 52.8 (100's of feet of gutter per mile) x (constituent concentration/1000). E.g., the rate of BOD accumulation on an urban area that is single family residential is: 0.7 x (2 x 52.8) x 5.0/1000 = 0.36 lbs/day/mile. Rates of pollutant buildup in pounds per day per mile are given in Table 4.

TABLE 4
Average Daily Loads Per Mile of Street (Chicago, Illinois)

Land Use	Average Load: lbs/day/mile of street			
	BOD	COD	N	PO4
Single Family Residential	0.36	2.95	0.03	0.004
Multiple Family Residential	0.87	9.70	0.15	0.012
Commercial	2.70	13.6	0.14	0.024
Industrial	1.45			

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*See Reference 5.

Comparison of the values in Tables 2 and 4 indicates that while the Chicago data is consistently lower than the Tulsa data, the rates of buildup are similar in commercial areas except for PO4. Multiple family residential values for the Chicago area also compare well with the Tulsa data except for PO4. The principal reason for the lower values in the Chicago data is that the reported values are for the soluble portion of the constituents and do not include that portion found in suspended and settleable material. Also, the Tulsa data are approximations based on over-simplified computations. Thus, of the two sets of reported data the Chicago data is probably better, given that the amount of constituents contained in the solids can be determined.

One fact that is quite evident from both the Chicago and Tulsa data is that the rate of buildup of pollutants on an urban watershed varies significantly with land use. Intuition would tell us this is true. Both sets of data indicate that industrial and commercial areas are much dirtier than residential areas. This would be expected since there is higher pedestrian and vehicular traffic densities in these areas. The data shows that pollutant accumulation rates are approximately one and one-half to five times as great in commercial and industrial areas as they are in residential areas.

ENTRY OF POLLUTANTS INTO URBAN RUNOFF

The first raindrops that fall on an urban watershed simply wet the land surface. As additional rain falls the impervious surface will become wet enough that some of the water begins to form puddles, filling the depression storage. This initial rain begins to dissolve the pollutants in the gutters, streets, and on other impervious surfaces and eventually, as this water actually begins to flow off the watershed it carries the dissolved material in it.

As rainfall intensity increases, overland flow velocities become sufficient to pick up solids. Suspended solids are, of course, picked up at smaller velocities than settleable solids. The settleable solids are carried off the watershed in two ways. If the velocity is sufficiently high, the settleable solids may be suspended in the overland flow. At lower velocities, particles may simply be rolled along the bottom surface toward the stormwater inlet.

The rain that initially falls on pervious surfaces infiltrates into the ground. If the rainfall is sufficiently intense, the infiltration capacity may be exceeded and the excess rainfall begins to fill the depression storage on the pervious surfaces. Finally, if the rainfall is of sufficient intensity and duration, runoff will begin to flow off the pervious areas, onto the impervious areas and thence into the stormwater inlets. Present experience, however, indicates that the amount of runoff, and hence the pollution loads, contributed from pervious surfaces in urban areas are small compared to those coming from the impervious areas and can be neglected in determining the quality of surface runoff. This is especially true of surfaces covered with vegetation such as lawns and gardens. Figure 6 illustrates the differences in runoff and pollution load from a watershed that would occur if it was converted from a park (90% pervious) into a multiple residential area (20% pervious).

ESTIMATION OF THE RATE OF POLLUTANT BUILDUP ON URBAN WATERSHEDS

Since dirt is the major component of street litter and is the primary source of pollutants in urban runoff, the most basic approach for estimating pollutant buildup rates would be to relate them to the dust and dirt accumulation rates.

Using APWA units (Table 3) for the rate of dust and dirt accumulation, the rate of buildup DD_L for a given land use L can be expressed as:

$$DD_L = dd_L \times (G_L/100) \times A_L \qquad (4)$$

where

DD_L = rate of dust and dirt accumulation on a watershed of land use L in lbs/day;

dd_L = rate of dust and dirt accumulation on watershed L in lbs/day/100 feet of gutter;

G_L = feet of gutter per acre in watershed L; and

A_L = area of watershed L in acres.

The rate factor dd_L should be supplied by the user for his area. Default values, which are those shown in Table 3 are incorporated into STORM and can be used if no better data are available.

The initial quality of a pollutant p on watershed L at the beginning of a storm can then be computed as:

$$P_p = (F_p \times DD_L \times N_D) + P_{po} \qquad (5)$$

where

P_p = total pounds of pollutant p on the watershed L at the beginning of the storm;

F_p = pounds of pollutant p per pound of dust and dirt;

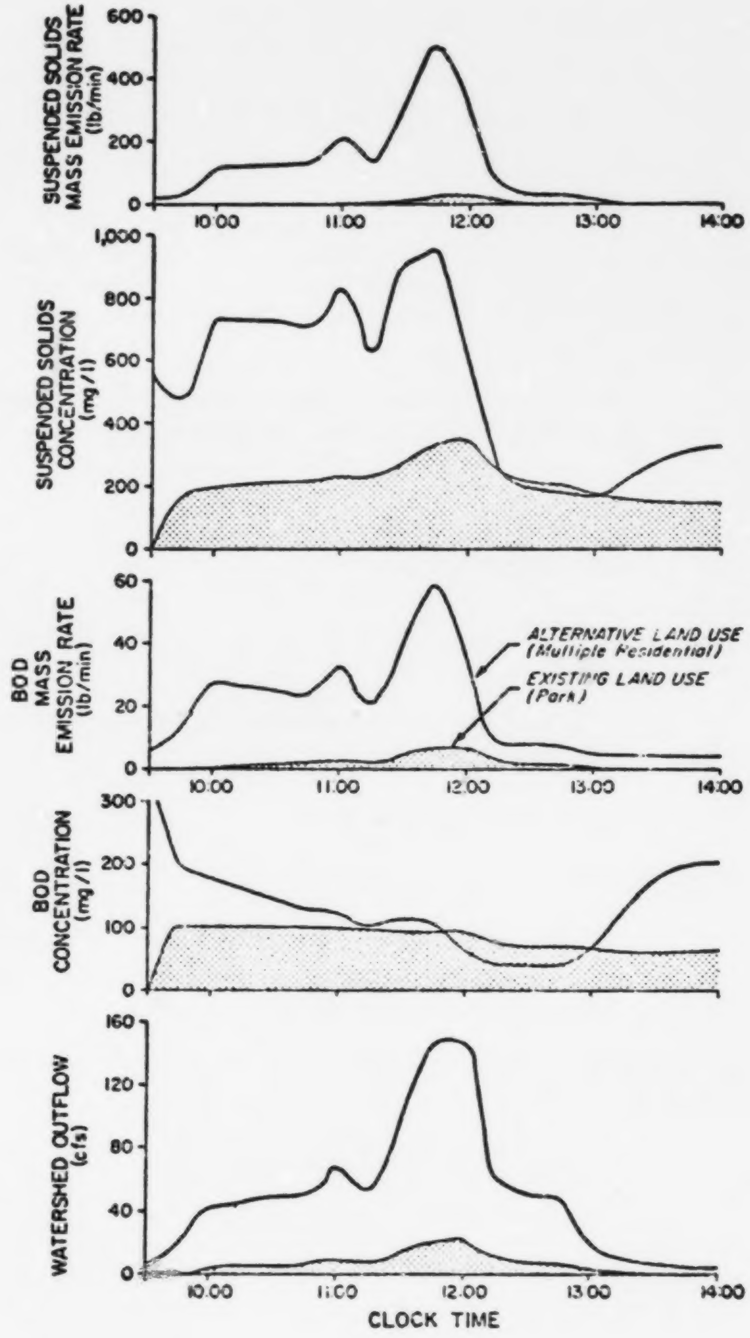

FIGURE 6
Effect of Changed Land Use on Characteristics
Of Subcatchment Runoff, Selby Street

N_D = number of dry days since the last storm, and

P_{po} = total pounds of pollutant remaining on watershed L at the end of the last storm.

In practice, P_p is usually limited to the amount that would be accumulated in a 90-day dry period. The reason for this is that the efficacy of extrapolating daily buildup rates beyond this point (which was arbitrarily selected) is uncertain. Moreover, if equation (5) is used repetitively over long periods of time, positive errors could tend to accumulate in P_{po} resulting in overly large values of P_p.

If street sweeping is practiced on the watershed, the number of dry days since the last storm must be modified to account for the number of street sweepings that occurred since the last rainfall. The correct expression to use for P_p is then:

$$P_p = P_{po}(1-E)^n + N_S \times DD_L \times F_p \left[(1-E) + \ldots (1-E)^n\right] + DD_L \times F_p \, (N_p - n\,N_S) \quad (6)$$

where

N_S = number of days between street sweepings;

n = number of times the street was swept since the last storm; and

E = efficiency of street sweeping (0.6 to 0.95).

DETERMINATION OF URBAN RUNOFF POLLUTION LOADS

To compute the amount of pollutant washed off the watershed during a storm, it is assumed that the amount of pollutant removed at any time t is proportional to the amount remaining:

$$\frac{dP_p}{dt} = -KP_p \quad (7)$$

We stated earlier that the runoff rate Q also affects rate of pollutant removal, therefore K must be functionally dependent upon Q. However, given two identical watersheds except for their area size, for the same rainfall rate r on both watersheds a higher runoff rate would occur from the larger watershed. This area effect can be eliminated by dividing the runoff Q by the impervious area of the watershed. The impervious area is used because only a negligible amount of the runoff comes from the pervious area. Since cfs per acre are equivalent to inches per hour, we can say that K is functionally dependent on the runoff rate R from the impervious area, where R is in inches per hour. Finally, assuming that K is directly proportional to R and that a uniform rainfall of 1/2 inch per hour would wash away 90 percent of the pollutant in one hour (a somewhat arbitrary assumption), we can say that K = 4.6R. Making this substitution into equation (7) and integrating over a time interval Δt (during which R is held constant) gives:

$$P_p(t + \Delta t) = P_p(t)e^{-4.6R\Delta t} \tag{8}$$

Equation (8) is the basic form of the overland flow quality model developed by Metcalf & Eddy, Inc., as part of the EPA Stormwater Management Model [2]. Although it is simplistic and contains many assumptions, it is the best overland flow water quality predictor or simulation model that presently exists. Moreover, experience with that model (See Reference 6 and Volume II of Reference 2) has shown it to give fairly good results.

Some idea of how equation (8) behaves can be gained by examination of Figures 7 and 8. Figure 7 shows that for a constant runoff rate R, the amount of pollutant remaining on the watershed decays exponentially. Under a time varying R the picture is quite different, as illustrated by the upper graph Figure 8. From the curve of P vs. t it can be seen that the amount of pollutant removed during an interval Δt is $P(t)-P(t+\Delta t)$. The rate of removal of mass from the watershed M_p is simply $[P(t)-P(t+\Delta t)]/\Delta t$, which can be expressed as:

$$M_p = P(t) \times (1-e^{-4.6R\Delta t})/\Delta t \tag{9}$$

The variation of M_p with time for the associated hydrograph is plotted in the lower graph of Figure 8. A plot of M_p versus t is termed a pollutograph, one of the most informative methods for expressing the pollutant load carried by urban runoff. To determine the concentration of a pollutant in the runoff as a function of time, one simply divides the pollutograph value M_p by Q (with appropriate conversion factors).

Equation (9) must be modified, however, because not all of the dust and dirt on the watershed is available for inclusion in the runoff at a given time t. Thus pollutants which are tied to the dust and dirt are not all available either. The Storm Water Management Model study [2] found that for suspended solids the available fraction at any time was:

$$A_{sus} = 0.057 + 1.4R^{1.1} \tag{10}$$

For settleable solids it has been assumed that the availability factor is

$$A_{set} = 0.028 + 1.0R^{1.8} \tag{11}$$

With regard to BOD, nitrogen and phosphate, recall that the APWA data [5] described the dissolved fraction, which is independent of the amount of

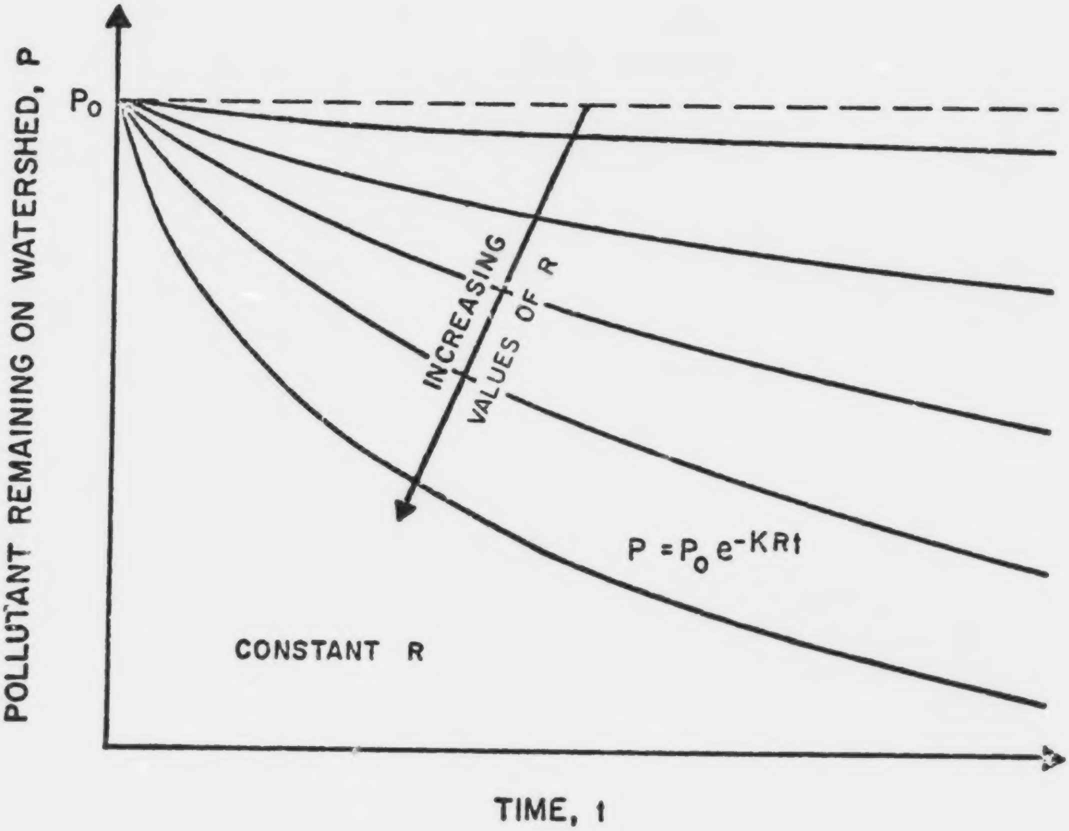

FIGURE 7
Basic Form of the Overland Flow Quality Model

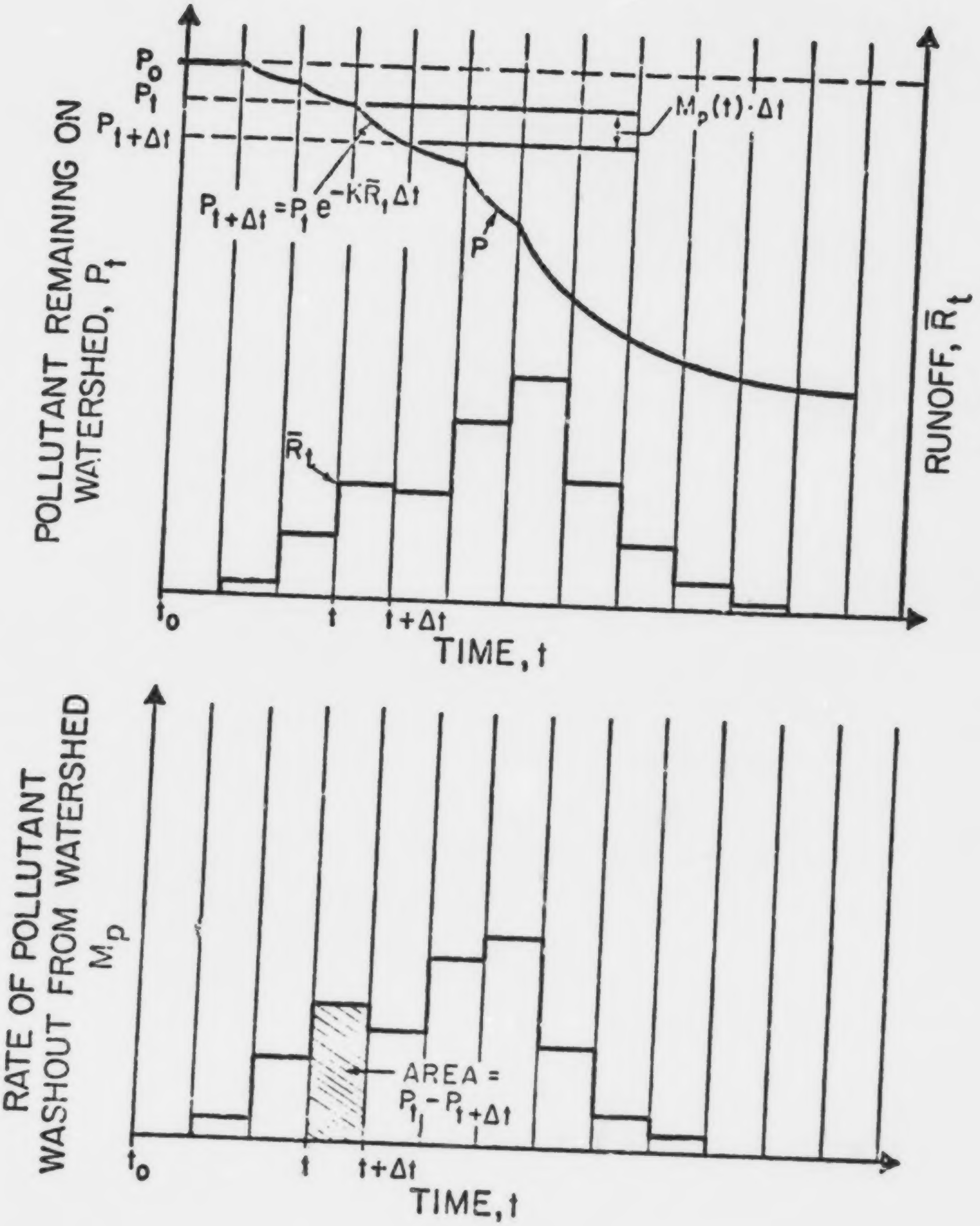
POLLUTANT REMAINING ON WATERSHED, P_t
P_o
P_t
$P_{t+\Delta t}$
$M_p(t) \cdot \Delta t$
$P_{t+\Delta t} = P_t e^{-k\bar{R}_t \Delta t}$
P
$\bar{R}_t$
RUNOFF, $\bar{R}_t$
t_o
t
$t+\Delta t$
TIME, t
RATE OF POLLUTANT WASHOUT FROM WATERSHED M_p
AREA = $P_t - P_{t+\Delta t}$
t_o
t
$t+\Delta t$
TIME, t

FIGURE 8
Development of Pollutograph (M_p vs. t)
From Time History of P_t

solids available for runoff. In the Storm Water Management Model study it was found that the BOD associated with the suspended solids was about 10 percent of the suspended solids load. We have further assumed that the BOD tied to the settleable solids is 2 percent of the settleable solids. For nitrogen and phosphate, we have assumed that BOD, N, and PO4 are associated with suspended and settleable solids in the same proportion as they are in the dissolved state.

Thus, correcting equation (9) for available suspended and settleable solids and adding the BOD, N and PO4 found in the solids, we get the following set of equations which are used in STORM:

Suspended Solids

$$M_{sus}(t) = A_{sus} P_{sus}(t) \times EXPT \qquad (12)$$

where

$A_{sus} = 0.057 + 1.4R^{1.1}$

$EXPT = (1-e^{-4.6R\Delta t})/\Delta t$, with $\Delta t = 1$ hour

Settleable Solids

$$M_{set}(t) = A_{set} P_{set}(t) \times EXPT \qquad (13)$$

where

$A_{set} = 0.028 + R^{1.8}$

BOD

$$M_{bod}(t) = P_{bod}(t) \times EXPT + 0.10\, M_{sus} + 0.02\, M_{set} \qquad (14)$$

Nitrogen

$$M_{nit}(t) = P_{nit}(t) \times EXPT + .045\, M_{sus} + .01\, M_{set} \qquad (15)$$

PO4

$$M_{PO4}(t) = P_{PO4}(t) \times EXPT + .0045\, M_{sus} + .001\, M_{set} \qquad (16)$$

SECTION 4
COMPUTATION OF TREATMENT, STORAGE AND OVERFLOW

PROCEDURE

Computations of treatment, storage and overflow proceed in an hourly step-by-step method throughout a period of rainfall/snowmelt record. For every hour in which runoff occurs the treatment facilities are utilized to treat as much runoff as possible. When the runoff rate exceeds the treatment rate, storage is utilized to contain the runoff. When runoff is less than the treatment rate, the excess treatment rate is utilized to diminish the storage level. If the storage capacity is exceeded, all excess runoff overflows into the receiving waters and does not pass through the storage facility. This overflow is lost from the system and cannot be treated later. While the storm runoff is in storage its age is increasing. Various methods of aging are used including average, first-in: last-out, first-in: first-out, or others, depending on the physical conditions encountered.

The computation of storage and the interplays among rainfall/snowmelt, storage and treatment represent a simplistic approach for dividing a rainfall record into unique events such that the event is defined in terms of the urban system. For example, whether two "storms" are considered as two isolated occurrences or as one large storm is entirely dependent upon how the system will react to them. If the system has not recovered from the first when the second arrives, the two definitely will interact and hence must be considered together. "Events" are defined as beginning when storage is required and continues until the storage reservoir is emptied. All the rainfall occurring within this period is regarded as part of the same event. If precipitation produces runoff that does not exceed the treatment rate, the runoff will pass through the treatment process but will not register as an event. From the standpoint of the urban stormwater system, such precipitation is inconsequential and hence is not part of an "event" even if it should occur immediately preceding an obvious event.

The runoff coming into the storage/treatment system is given by equation (1). The quantity of system overflows are computed using:

$$Q_o = R - Q_T - Q_S \qquad (17a)$$

$$Q_T = \text{minimum of } (R + Q_{s_{t-1}} , T) \qquad (17b)$$

$$Q_s = \text{minimum of } (R - Q_T , S) \qquad (17c)$$

where

Q_o = watershed runoff overflow, in inches;

Q_T = watershed runoff treated, in inches;

Q_s = watershed runoff stored, in inches;

$Q_{s_{t-1}}$ = watershed storage remaining in previous hour, inches;

R = watershed runoff as calculated using equation (1), inches;

T = treatment rate in watershed inches/hour; and

S = storage capacity in basin, inches.

The quality of system overflows are computed as follows for each pollutant for each hour:

$$M_{po} = M_p (Q_o / R) \tag{18}$$

$$M_{pT/s} = M_p - M_{po} \tag{19}$$

where

M_{po} = total pounds of pollutant overflowing from system;

M_p = total pounds of pollutant p coming into the system; and

$M_{pT/s}$ = total pounds of pollutant p going to storage/treatment.

The program does not model the treatment process but it does compute the quantity of water treated. It is assumed that the pollutants will be reduced to an acceptable level before the storm water is released. The age of pollutant in storage is computed as previously mentioned. The format of STORM output data is given in Table 5.

TABLE 5
"STORM" Output

I. STATISTICS BY EVENTS
   - A. RAINFALL
      - 1. *DURATION OF RAINFALL EVENT*
      - 2. *HOURS OF RAIN*
      - 3. *TOTAL RAINFALL*
   - B. STORAGE
      - 1. *TIME SINCE LAST EVENT*
      - 2. *DURATION OF STORAGE*
      - 3. *TIME TO EMPTY*
      - 4. *MAXIMUM STORAGE USED*
   - C. OVERFLOW
      - 1. *TIME OVERFLOW STARTS*
      - 2. *DURATION OF OVERFLOW*
      - 3. *QUANTITY OF OVERFLOW*
      - 4. *OVERFLOW IN FIRST THREE HOURS*
   - D. TREATMENT
      - 1. *DURATION OF TREATMENT*
      - 2. *QUANTITY TREATED*
   - E. QUALITY (susp. solids, sett. solids, BOD, nitrogen, phosphorous)
      - 1. *MASS EMISSION IN RUNOFF*
      - 2. *MASS EMISSION OF OVERFLOW*
      - 3. *MASS EMISSION DURING FIRST THREE HOURS OF OVERFLOW*

II. AVERAGE STATISTICS (A-E ABOVE)
   - A. FOR ALL EVENTS
   - B. FOR ALL OVERFLOW EVENTS
   - C. EVENTS / YR
   - D. OVERFLOWS / YR

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THE EPA STORMWATER MANAGEMENT MODEL

By

Wayne C. Huber*

James P. Heaney*

INTRODUCTION

An enormous pollution load is placed on streams and other receiving waters by combined and separate storm sewer overflows. It has been estimated that the total pounds of pollutants (BOD and suspended solids) contributed yearly to receiving waters by such overflows is of the same order of magnitude as that released by all secondary sewage treatment facilities (1,2). The Environmental Protection Agency (EPA) has recognized this problem and led and coordinated efforts to develop and demonstrate pollution abatement procedures (1). These procedures include not only improved treatment and storage facilities, but also possibilities for upstream abatement alternatives such as rooftops and parking lot retention, increased infiltration, improved street sweeping, retention basins and catch-basin cleaning or removal. The complexities and costs of proposed abatement procedures require that care and effort be expended by municipalities and others charged with decision making for the solution of these problems.

It was recognized that an invaluable tool to decision makers would be a comprehensive mathematical computer simulation program that would accurately model quantity (flow) and quality (concentrations) during the total urban rainfall-runoff process. This model would not only provide an accurate representation of the physical system, but also provide an opportunity to determine the effect of proposed pollution abatement procedures. Alternatives could then be tested on the model and least cost solutions could be developed.

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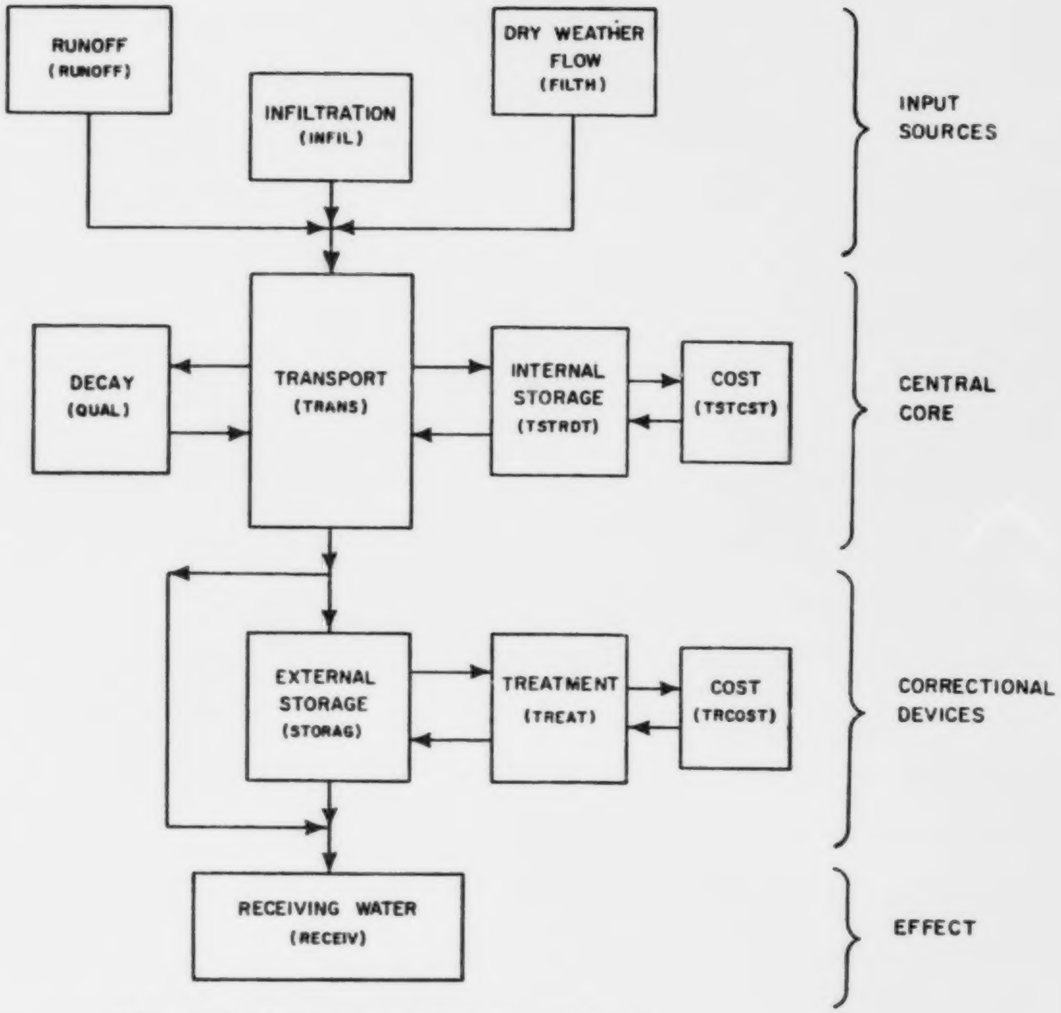
*Associate Professors. Environmental Engineering Sciences, University of Florida, Gainesville, Florida.

As a result, the University of Florida (UF), Metcalf and Eddy, Inc., Engineers (ME) and Water Resources Engineers (WRE) were awarded a joint contract for the development, demonstration and verification of the Storm Water Management Model (SWMM). The resulting model, completed in October, 1970, has been documented(3) and is presently being used by a variety of consulting firms and universities.

The present SWMM is descriptive in nature and will model most urban configurations encompassing rainfall, runoff, drainage, storage/ treatment, and receiving waters. The major components of the SWMM are illustrated in Figure 1 in which program segments are seen to correspond to physical components of the urban runoff process. The SWMM also has preliminary decision-making capabilities by virtue of cost estimation of abatement facilities, hydraulic pipe-sizing, and adaptation to optimization techniques for overall urban water resources planning.

Most of the theory behind the operation of the SWMM has remained unchanged since its publication in Volume I, Final Report of the


RUNOFF
(RUNOFF)
INFILTRATION
(INFIL)
DRY WEATHER FLOW
(FILTH)
INPUT SOURCES
DECAY
(QUAL)
TRANSPORT
(TRANS)
INTERNAL STORAGE
(TSTRDT)
COST
(TSTCST)
CENTRAL CORE
EXTERNAL STORAGE
(STORAG)
TREATMENT
(TREAT)
COST
(TRCOST)
CORRECTIONAL DEVICES
RECEIVING WATER
(RECEIV)
EFFECT

Note: Subroutine names are shown in parentheses.

Figure 1 OVERVIEW OF MODEL STRUCTURE

Storm Water Management MOdel (3). This document remains as the primary background reference for the underlying principles of the model, and material found in it is not repeated here.
However, many changes have been made in details of the program, input/output requirements and data descriptions. As a result. an official "Release 2" of the SWMM was issued in March 1975 (4).
The reader is referred to this publication which includes a revised and improved User's Manual to accompany the "Release 2" program.

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DECISION-MAKING FOR WATER QUANTITY AND QUALITY CONTROL

By

James P. Heaney*

This section presents the results of the analysis into alternative procedures for selecting the "optimal" overall stormwater control strategy. The word "optimal" is used in a mathematical sense to denote the alternative which is best for the problem as formulated. Much use has been made of such models during the past ten years. We have been actively involved in such developments. For example, Heaney (1968) developed a mathematical programming model to allocate water supply among competing users in the Colorado River Basin. Pyatt _et al_. (1969) and others have set up optimization models to determine the least cost combination of sewage treatment and low flow augmentation.

More recently, Kirshen, Marks, and Schaake (1972) have devised a mathematical model for analyzing stormwater quality control alternatives. A linear programming model is used to determine the sizes and operating policies for sewer pipes, storage reservoirs, and treatment plants. The SWMM is used to generate boundary conditions for the optimization model and to provide more detailed information on performance of the system.

The above optimization models assume some specified level of performance for the single-purpose system under consideration. Unfortunately, from a computational point of view, many of the stormwater quality control alternatives serve other functions such as drainage and flood control. Thus, the urban water quality problem seems to fit well into a more general framework of urban water resources management. An overall strategy

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*Associate Professor, Environmental Engineering Sciences, University of Florida, Gainesville, Florida.

for examining this more general problem has been developed. Addressing the problem in this broader perspective, the determination of the overall optimal solution can be achieved using one of several suggested procedures. A more innovative aspect of the analysis examines the questions of cost sharing among the various study areas within the urban area.

If one examines the various components of urban water resources, he is made aware of how each problem has been treated independently. Traditional concern for safe disposal of wastewater focuses on design of treatment units. There has been little systematic effort to devise overall strategies for drainage and flood control. The studies in Denver are a notable exception and they shall be discussed later. The question of urban water supply has been dealt with as a separate problem. Likewise, the related problems of recreation and open space have been analyzed separately from the other problems. As a result, it is difficult to gain insight as to how they all tie together.

Having analyzed these other problems, one begins to gain enthusiasm that perhaps "there is a better way." More enthusiasm was gained after participating in the layout of an 800-acre (3.24 Km^2) planned urban development which was an overall density of four units/acre and was able to satisfy the following constraints:

1) water use $\leq$ safe recharge of the aquifer;
2) on-site control of storm water from a 50-year storm;
3) no development of structures in the floodplain;
4) no direct discharge of storm water into on-site recreational lakes; and
5) disposal of treated sewage effluent onto golf course.

This area has about 86 percent of the total land in open space. The development is for middle income groups and comprises a mix of detached dwelling units, townhouses, apartments, and condominiums. It appears that the development costs are competitive or lower than existing practices.

The methodology used in addressing the problem can be partitioned into the several components listed below:

A) description of the study area;
B) selection of commodities to be analyzed;
C) inventory of sources;
D) inventory of controls;
E) economic analysis of control options;
F) specification of performance criteria;
G) statement of objectives; and
H) system optimization and examination of equity question.

Although detailed discussion of these components was presented in this Short Course, this information is also contained in the following technical report:

EPA-670/2-75-022
Urban Stormwater Management Modeling and Decision-Making
James P. Heaney
Wayne C. Huber
University of Florida

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Selected Case Studies Using Stormwater Management Models - Quantity Aspects

By

Gerald T. Orlob*

INTRODUCTION

The problems encountered in putting the Storm Water Management Models to practical use are best illustrated by "hands-on" experience--actual cases. While no particular set of case studies will highlight the myraid of unique problems that can be encountered in accord with Murphy's Law, we have chosen a set that shows at least some of the major obstacles to be overcome. These studies include investigations of:

1. Northwood Tract, Baltimore, Md.
2. Pegel T_1/T_2, City of Hamburg
3. View Ridge, City of Seattle
4. Vine Street, Melbourne, Australia
5. Stability Problems--A Special Study

In addition, some special problems encountered with wiers and "surge tanks" in storm drainage systems are discussed.

PROBLEMS ENCOUNTERED--POSSIBLE REMEDIES

Among the difficulties in adapting the models to practical use in simulation of storm runoff the following seem to be the most common:

1. Observed and simulated peaks of runoff are not coincident in time.

2. Watershed yields are not in agreement.

3. There is a lag between observed and simulated events.

4. Simulated runoff pattern, although similar in shape and area to that observed, tends to be smoother, more attenuated.

5. Simulated runoff is discontinuous.

6. Initial peaks are not well simulated although subsequent peaks are, and vice versa.

A few brief comments on each of these problem areas may help us to focus on some possible causes before we review the selected cases.

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*Senior Partner, G. T. Orlob & Associates, Orinda, California.

COINCIDENCE OF PEAKS

This may result from an incorrect start time for the model, from not offsetting the infiltration process in the initial stages of the storm, or from an incorrect estimation of basin slope or imperviousness, or both. Start time and basin characteristics may be adjusted; the offset in infiltration simulation is conceptual and would have to be corrected in the model.

WATERSHED YIELD

This discrepancy between model and prototype is a most common problem. It may result from improper estimation of actual rainfall on the basin, incorrect assessment of long-term infiltration, or incomplete gaging of runoff. There is no substitute for good precipitation data or careful hydrologic analysis. Infiltration estimates can be adjusted by modifying Horton's equation, i.e., f_c, the final constant value of infiltration.

LAG

In addition to incorrect start time and poor estimation of basin parameters, lag may be caused by over-application of other time dependent processes, e.g., infiltration, depression filling, etc. Adjustment of the infiltration decay rate or the amount of depression storage may correct this difficulty.

ATTENUATION

Excessive attenuation of the runoff hydrograph may be attributed to using too long a time step in integration, too coarse a structure for representing basin detail, or over-application of storage routing. Usually, better results are obtained by increasing detail, both spatially and temporally. The inverse problem, i.e., excessive sharpening of peaks and valleys, may be a result of failure to consider routing effects in overland flow and conduits.

DISCONTINUOUS RUNOFF

Excessive long-term infiltration may cancel out precipitation contribution to runoff. Failure to properly represent pervious areas can result in a pattern where runoff ceases too abruptly. A remedy is found in adjusting infiltration rates for pervious areas.

INITIAL PEAKS

Poor representation of initial peaks is most likely related to overcorrection for depression storage, interception, or initial infiltration. Occasionally, for large basins, it can result from the non-uniformity of precipitation associated with a moving storm. In the latter case, a remedy is to lag input precipitation to a more detailed basin representation.

CASE STUDIES

Northwood Tract

Baltimore's Northwood Tract has served as an excellent study example for calibration and verification of the EPA SWMM and WRE's modifications of this model package. The Tract, illustrated in Figure 1, encompasses approximately 47.4 acres of urban drainage with an average imperviousness of 68 percent. The collected flow from the area passes through a 12-foot Parshall Flume with a rated capacity of about 400 cfs. Two continuous recording rain gages are available, one of the weighing bucket type in the northeast quadrant of the area and the other of the tipping bucket type near the southeast corner of the area.

Figures 2, 3, and 4 illustrate the performance of the SWMM, as modified by WRE, in simulating the runoff generated by three storms over the area. Figure 2 may be considered the calibration run and Figure 3 a verification check. Figure 4 illustrates the model's capability to simulate a complex storm of highly variable temporal pattern.

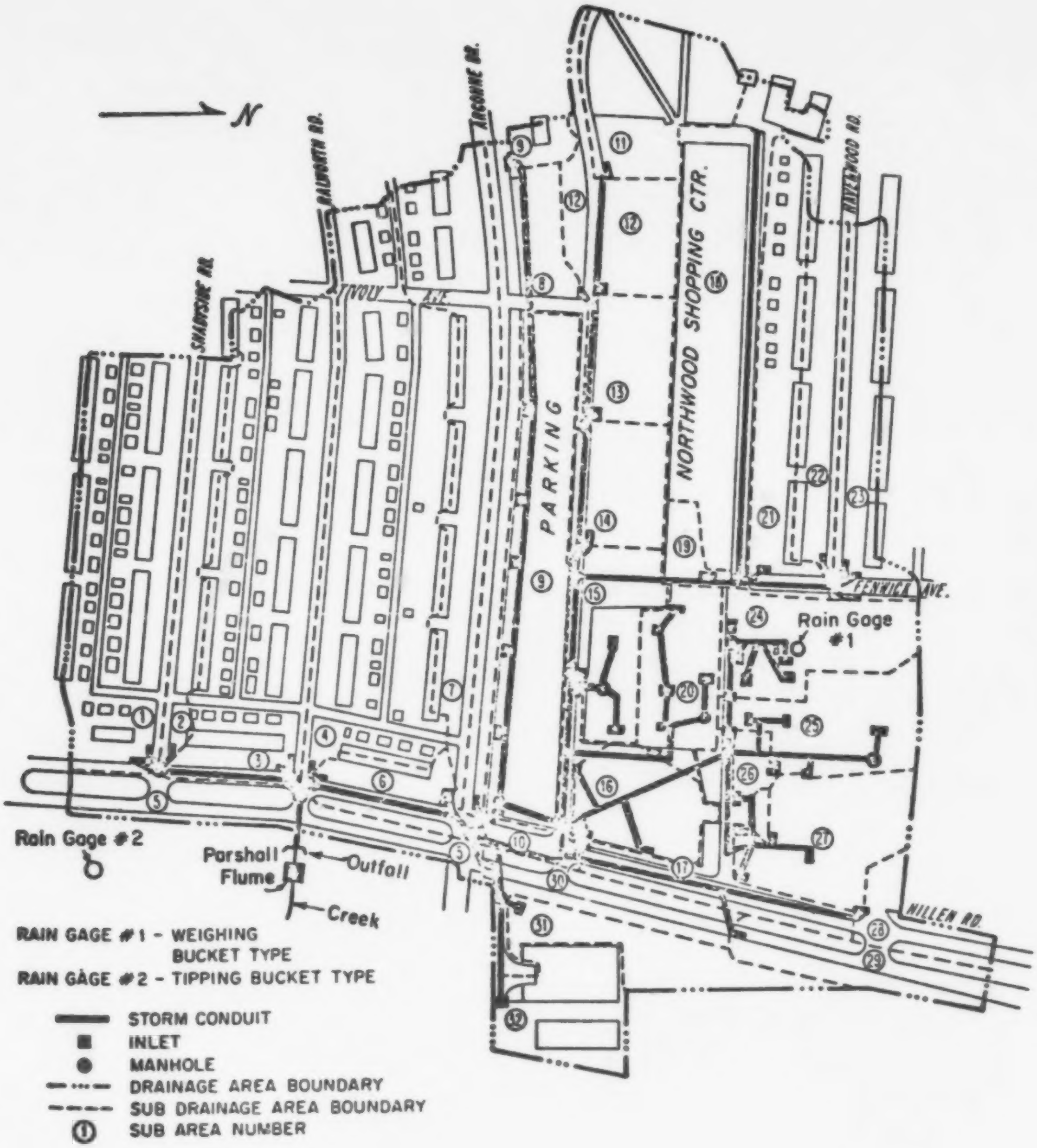
Two specific problems were encountered in these studies. First, there was a tendency for the model to produce accentuated peaks early in the storm, while good checks were made in the balance of the simulation. This was corrected in this example by increasing the allocation to detention storage early in the simulation. Second, the model failed to simulate the very low flows apparently observed at the Parshall Flume midway through the storms of 1 August and 4 August (second storm). Upon examining the data it was determined that this was primarily due to inaccuracies in low flow measurement in the very wide-throated flume.

In general, however, the simulations are quite excellent and give a good account of the model's capabilities.

Pegel T_1/T_2, City of Hamburg

The Storm Water Management Models have been adapted for a unique study of the urban drainage system for the City of Hamburg, Germany. Figure 5, A and B, shows the configuration of the test basin and the "testnetz", designated by the station at the lowest point, Pegel T_1/T_2. Results of simulation, following a calibration phase are summarized in Figure 6. The computed runoff ("Berechnet") compares well with the measured ("Gemessen") flow for a storm with two distinct precipitation peaks.

Some special problems were encountered in calibration of the model, however, and a few changes in the program were made to make the model more realistic. One of these changes is illustrated in Figure 7, where a "before" and "after" comparison is made. In the initial version of the runoff model, referred to here as "Selby Street" after that used in the San Francisco Study, runoff was delayed until the full detention flow was developed over the drainage area. This resulted in a delay in the rising limb of the hydrograph that was not realistic. Actually,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SHADYSIDE RD.
DALSWORTH RD.
ARGONNE DR.
TIVOLY AVE.
PARKING
NORTHWOOD SHOPPING CTR.
HAVENWOOD RD.
FENWICK AVE.
HILLEN RD.
Rain Gage #1
Rain Gage #2
Parshall Flume
Outfall
Creek
RAIN GAGE #1 - WEIGHING BUCKET TYPE
RAIN GAGE #2 - TIPPING BUCKET TYPE
STORM CONDUIT
INLET
MANHOLE
DRAINAGE AREA BOUNDARY
SUB DRAINAGE AREA BOUNDARY
SUB AREA NUMBER
1
2
3
4
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Figure 1. PLAN OF NORTHWOOD DRAINAGE AREA BALTIMORE, MARYLAND

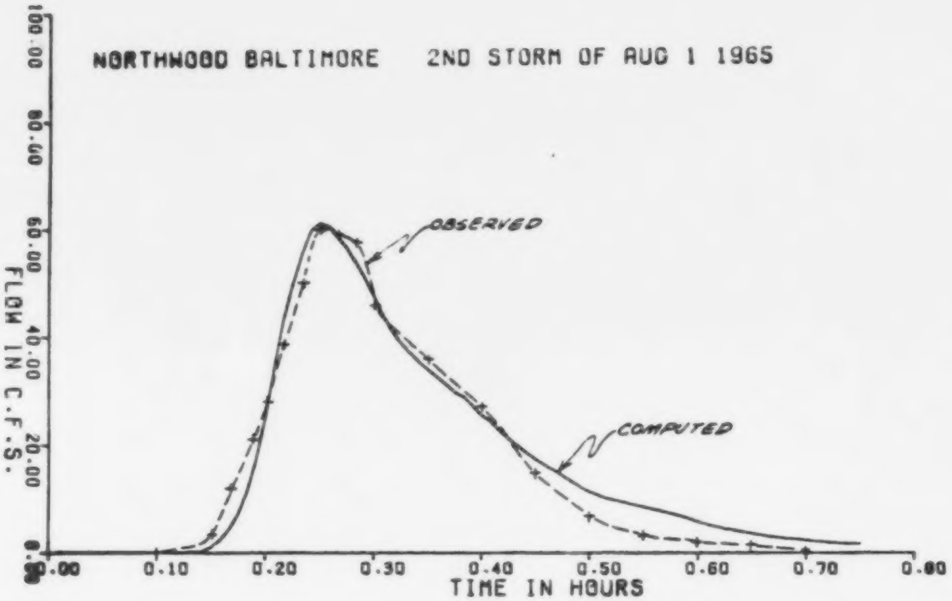


Figure 2. RUNOFF SIMULATION, BALTIMORE NORTHWOOD TRACT
2nd STORM OF AUG. 1, 1965

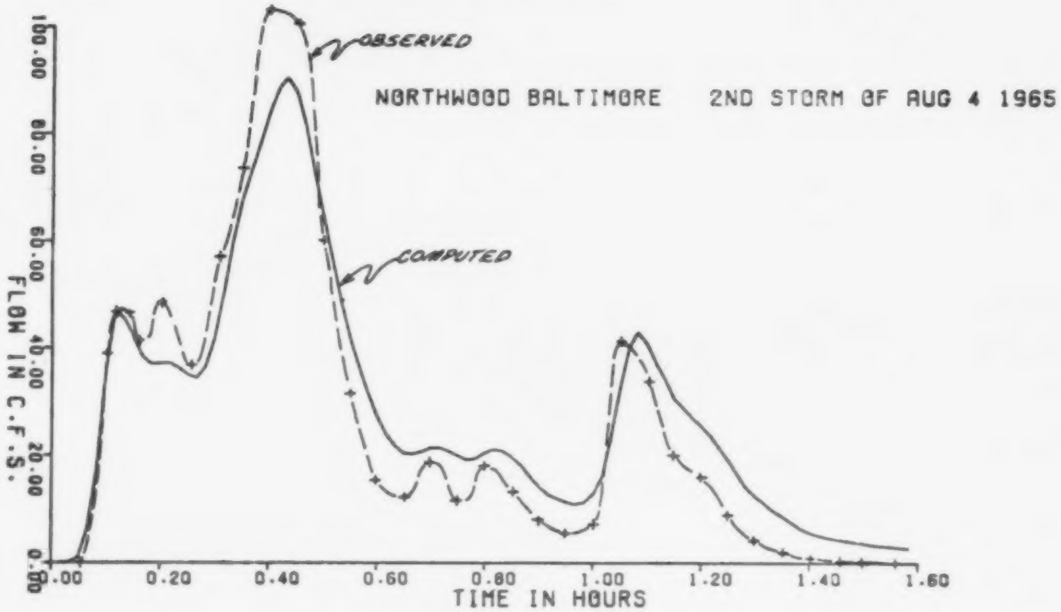


Figure 3. RUNOFF SIMULATION, BALTIMORE NORTHWOOD TRACT
2nd STORM OF AUG. 4, 1965

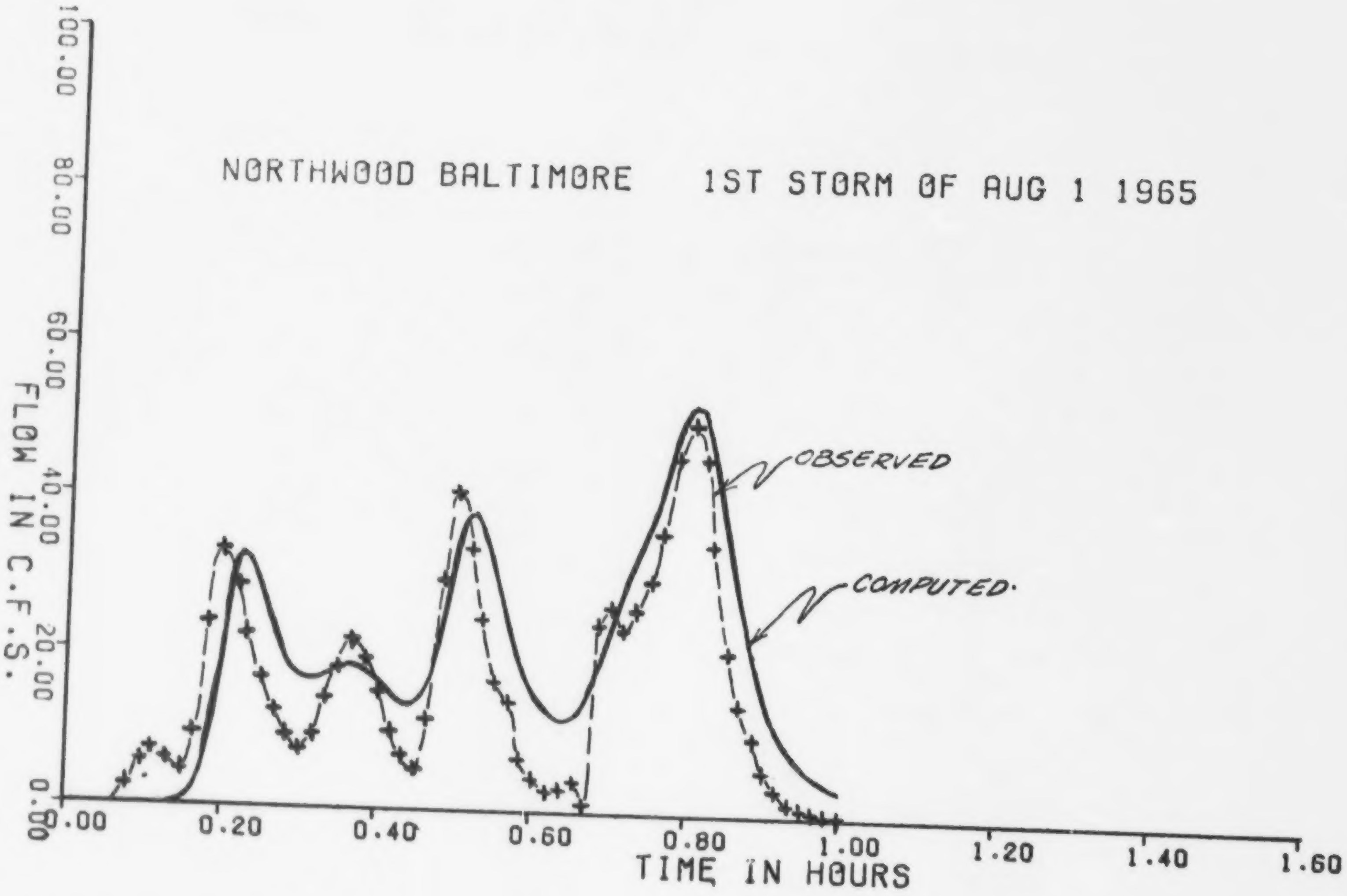
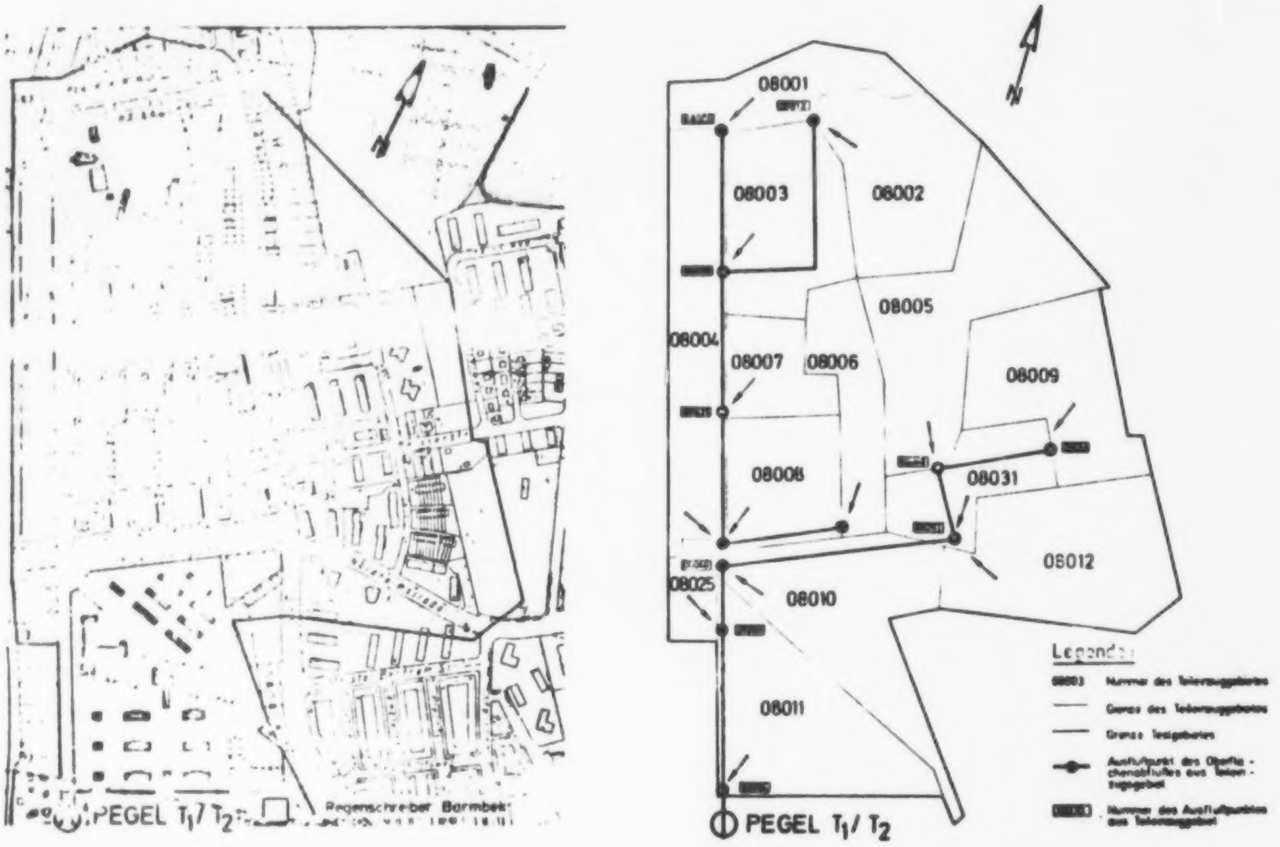


Figure 4. RUNOFF SIMULATION, BALTIMORE NORTHWOOD TRACT

1st STORM OF AUG. 1, 1965


PEGEL T_1/T_2
Regenschreiber Barmbek
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08011
08012
08025
08031
PEGEL T_1/T_2
Legende

Figure 5. TEST AREA AND "TESTNETZ," PEGEL T_1/T_2, CITY OF HAMBURG (Ref. 2)

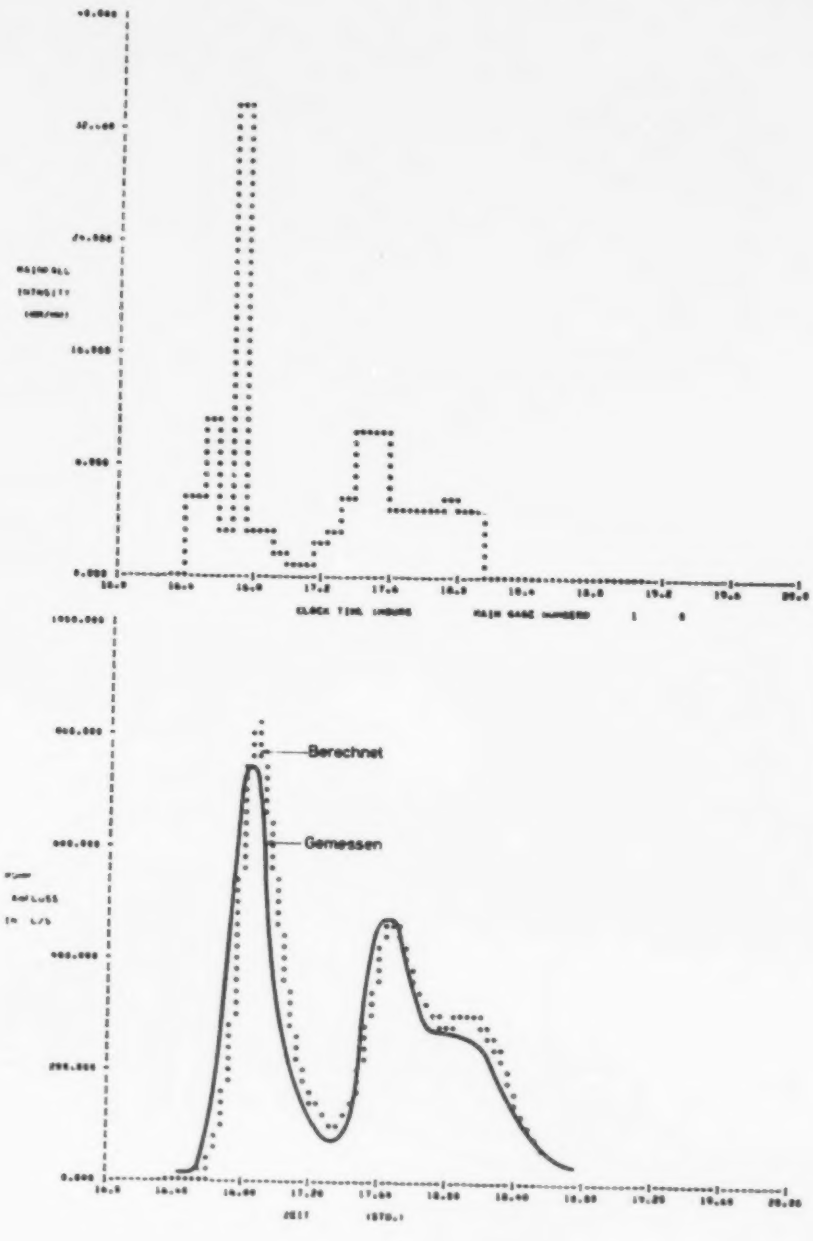

Figure 6. SIMULATION OF STORMWATER RUNOFF
PEGEL T_1/T_2, CITY OF HAMBURG (Ref. 2)

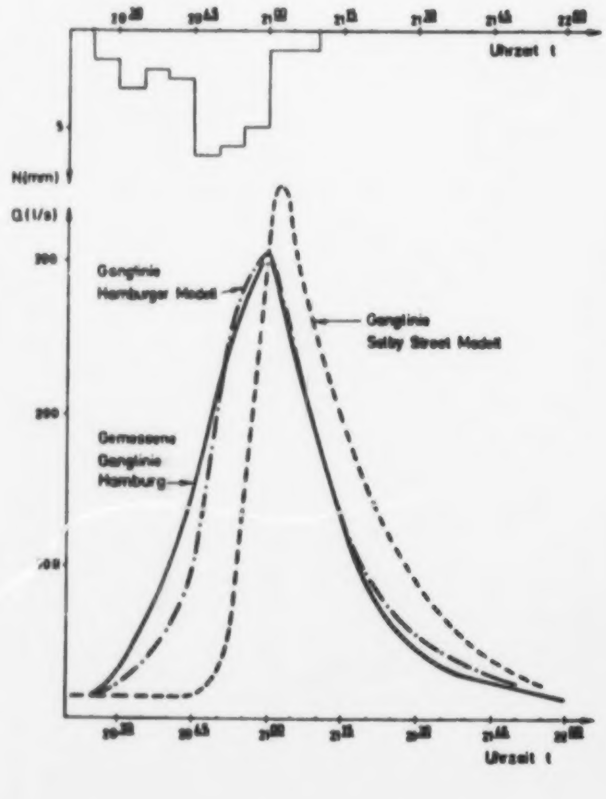

Figure 7. COMPARISON BETWEEN HAMBURGER AND SELBY STREET MODELS (Ref. 2)

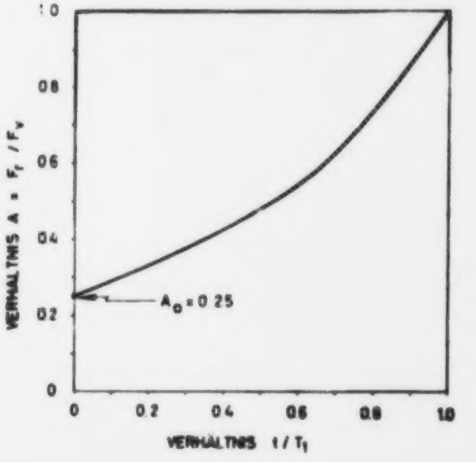

Figure 8. IMPERVIOUS RUNOFF FACTOR (Ref. 2)

runoff should become tributary to the gaging point shortly after rainfall commences, i.e., after initial wetting and some surface detention is satisfied, but not all. This was corrected for by allowing some fraction of the flow from the impervious area to be immediately contributing and that fraction to increase steadily thereafter. Figure 8 shows the nature of the function employed. It has the general form

$$A_t = A_o + (1 - A_o) \frac{t}{T_1} e^{k(1 - \frac{t}{T_1})}$$

where A_t is the fraction of calculated surface runoff from impervious areas that are fully tributary and t/T_1 is the fraction of the time required to reach full surface detention. An initial value, A_o, can be assumed, say 0.2 to 0.4,(depending on the nature of the drainage surface and the shape of the curve of A vs t/T_1) can be accommodated by the coefficient k, usually taken in the Hamburg study as unity.

The result of making this correction is apparent in Figure 7 where the Hamburger Model is seen to agree closely with the measured value. The Selby Street version is seen to produce a hydrograph displaced generally to the right, i.e., delayed from the measured pattern.

View Ridge and Central Business District, Seattle

Figure 9 shows the location of a fully urbanized residential area, View Ridge, in Seattle, Washington, that was used as a test case for model calibration in connection with an urban drainage study performed by Water Resources Engineers and Kramer, Chin and Mayo, Inc. for the Seattle District Corps of Engineers. The area is 605 acres and the average imperviousness is 40 percent.

As one would expect, some difficulties were experienced in adjusting rainfall data from the raingage several miles away to the drainage basin. Also, the basin terrain is generally hilly, whereas the gage is situated in the somewhat flatter open area of the University of Washington campus. Nevertheless, after several adjustments of the model an acceptable calibration was obtained. This is shown in Figure 10 together with a similar test for a small (25 acre), relatively impervious (99 percent) business area in downtown Seattle.

It is noted that the model gives a good account of the peak flow for the View Ridge catchment, but is a bit too low in the trailing portion of the storm. For the Central District the reverse seems to be the case, although we note that an adjustment of the peak upward, perhaps by modifying surface detention, would probably result in a comparison similar to that for View Ridge. Apparently, in both cases, too much weight is given to sustained runoff from already wetted areas.

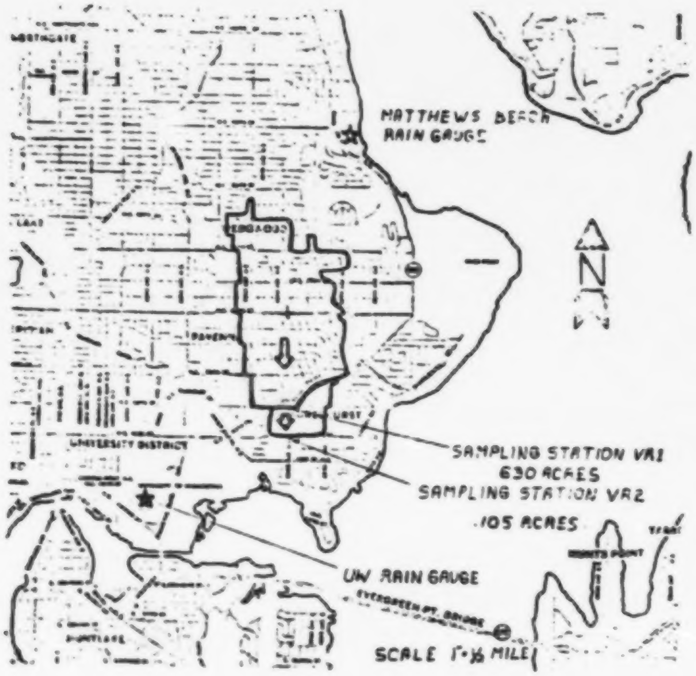
MATTHEWS BEACH
RAIN GAUGE
N
UNIVERSITY DISTRICT
SAMPLING STATION VR1
630 ACRES
SAMPLING STATION VR2
105 ACRES
UW RAIN GAUGE
EVERGREEN PT. BRIDGE
SCALE 1"=½ MILE

Figure 9. VIEW RIDGE URBAN DRAINAGE BASIN

Reproduced from
best available copy.

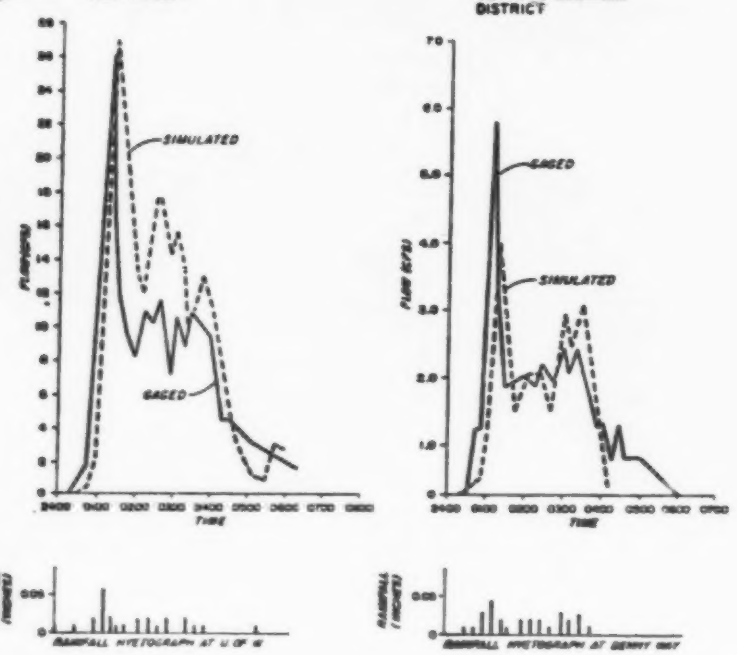

Figure 10. CALIBRATION OF RUNOFF MODEL, SEATTLE DRAINAGE STUDY
(Ref. 3)

Vine Street, Melbourne, Australia

Heeps and Mein (4) carried out an excellent comparative study of the EPA-SWMM and models developed by the Road Research Laboratory in England (5) and University of Cincinnati (6). The three models were tested against gaged runoff from two urban watersheds under a variety of conditions. Among the model capabilities and properties tested were:

a. ability to simulate runoff from storms of diverse characteristics,

b. effect of degree of discretization,

c. effect of depression storage,

d. effect of infiltration,

e. treatment of pervious areas, and

f. sensitivity to input data.

An evaluation matrix was selected for purposes of comparisons; it included percent of observed runoff, percent of observed peak, sum of squares of errors, and processor time. It was generally concluded that the EPA-SWMM was the superior in performance of the three model packages tested although it was noted that processor time was invariably greater for the SWMM.

To illustrate the effect of discretization each model was tested for the same storm conditions on the Vine Street Basin at two or three levels of detail. Figure 11 illustrates three levels of discretization employed. In the upper sketch the 70 hectare catchment is divided into 12 subbasins ranging in size from 0.1 to 18.6 ha. In the middle sketch the catchment is divided into 51 subbasins and in the lower sketch, the maximum discretization there are 256 subbasins. The detail of inlets and drains was increased roughly in proportion.

Figure 12 shows some results of the SWMM for coarse and medium discretization. It is noted that the major effect of increased detail is a reduction in peak flow, due primarily to increased conduit routing and corresponding attenuation of the runoff hydrograph.

Figure 13 illustrates a comparison of the three models for a fairly complex storm over the Vine Street Catchment. The SWMM is seen to be the superior model for this test. The RRL model tends toward excessively high peaks and the CURM, while giving a good account of the early peak, fails to simulate the balance of the runoff pattern. This is attribute by Heeps and Mein to neglect of the runoff contribution from pervious areas. The comparison statistics for this test of the three models are:

	SWMM	CURM	RRL
% Observed Runoff	102	139	49
% Observed Peak	106	189	91
Sum of Squares of Errors (m^6/sec^2)	61	78	86
Processor Time (min.)	34.23	31.05	1.87

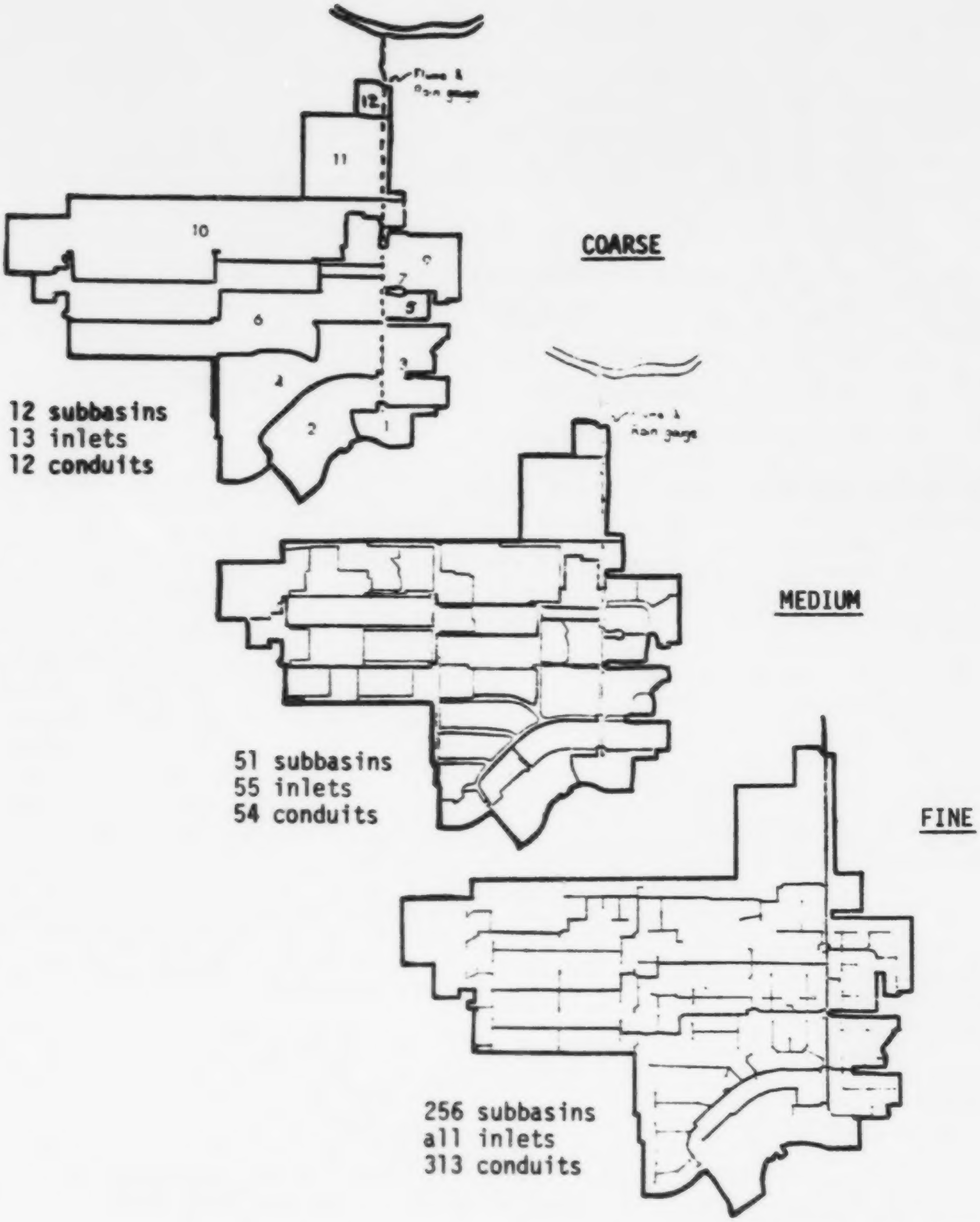
Flume & Rain gage
12
11
10
COARSE
9
7
5
6
3
4
2
1
12 subbasins
13 inlets
12 conduits
Flume & Rain gage
MEDIUM
51 subbasins
55 inlets
54 conduits
FINE
256 subbasins
all inlets
313 conduits

Figure 11. VINE STREET CATCHMENT
MELBOURNE, AUSTRALIA
(after Heeps and Mein, Ref. 4)

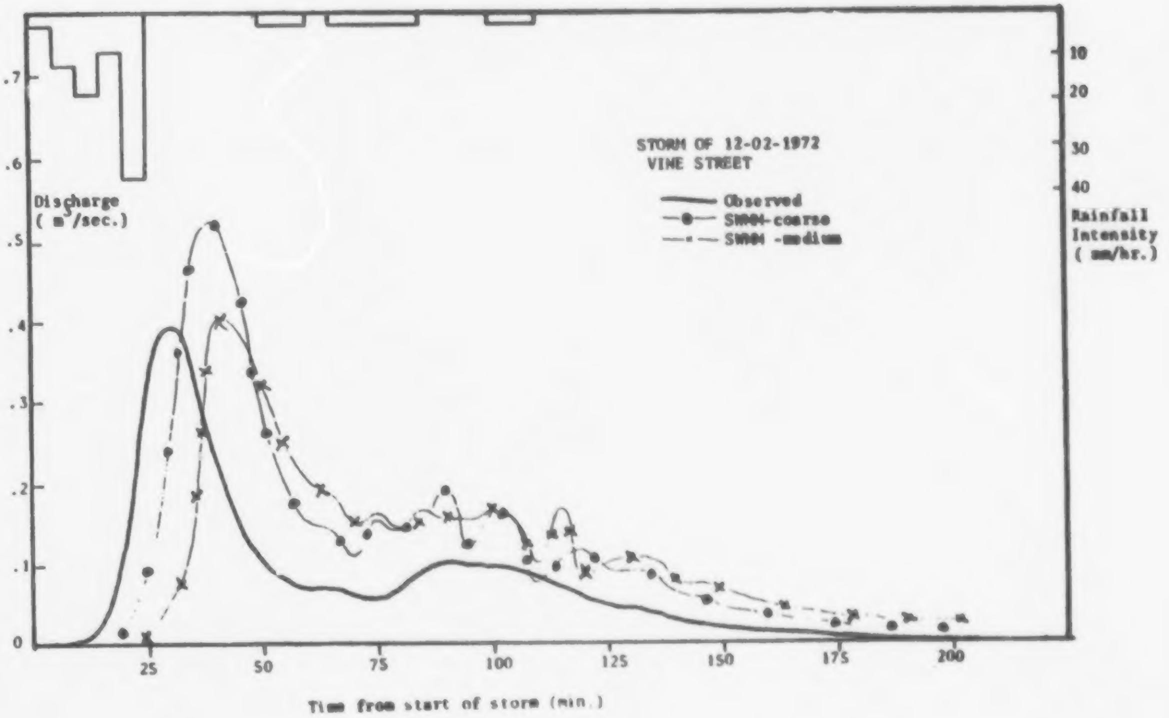

Figure 12. PERFORMANCE OF EPA-SWMM ON VINE STREET CATCHMENT AT COARSE AND MEDIUM LEVELS OF DISCRETIZATION (Ref. 4)

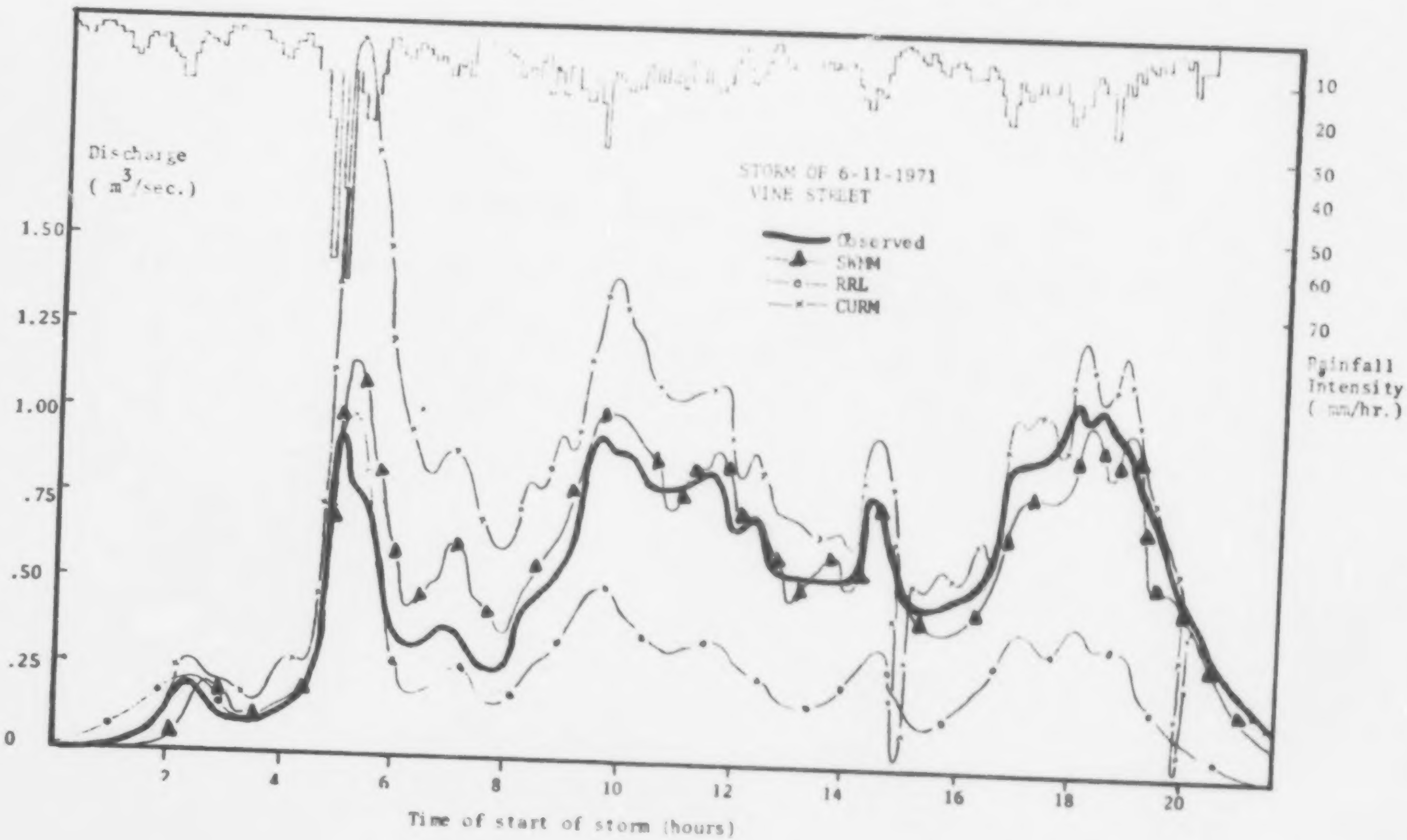

Figure 13. COMPARISON OF SWMM, CURM, AND RRL MODELS, VINE STREET CATCHMENT (Ref. 4)

Stability Problems--A Special Study

Simulation of the dynamics of storm flow through closed conduits is fraught with possibilities for computational instability. Often, however, it is difficult to distinguish numerically induced oscillations in the solution from hydraulic behavior that may sometimes produce real fluctuations, even reversals, in flow. The usual quick test for computational instability is to determine whether or not the oscillation strengthens with time, i.e., diverges, damps out, i.e., converges, or remains steady in period and amplitude.

In the first instance computational instability is most likely. One must devise methods for eliminating it. These include changes in time step, averaging, or changes in computational techniques, among the more prominent approaches.

In the second case, solution may be inefficient, but it may ultimately converge. Unfortunately, in simulation of transient phenomena these oscillations if damped slowly may musk the true nature of the solution; resulting in a worthless simulation. In such cases, one must seek means for accelerating convergence or removing oscillations from the solution entirely.

In the third case, the mere fact of repetitive oscillation may be indicative of a real phenomenon, for example, surge in a conduit induced by tidal oscillations at an outfall location. One must be assured, however, that the oscillations are pure, however, and not confused by errors introduced by the numerics of the solution method.

Some simple examples of numerical instability in application of the SWMM are illustrated in Figure 14 and 15. In the lower section of Figure 14, water levels at a manhole vary widely and in more or less a divergent manner when the time step of integration is 20 seconds. Reducing the time step to 10 seconds allows a steady progression to a converged solution (H = 1000.406 ± 0.0002), as may be seen in the upper portion of the figure. Similar, although more dramatic, is the experience in a conduit connecting to the manhole. In the lower section of Figure 15 the flow is seen to vary from almost zero to more than 16 cfs with an average of about 6.0 cfs. There is no distinctive pattern that suggests convergence. Contrariwise, the fluctuations are rather random in amplitude. However, when the time step is reduced to 10 seconds the flow settles down rather quickly to a value of about 5.959 ± 0.004 cfs. Note that while there are oscillations of about the same period as in the lower plot, these tend to reduce in amplitude with time. Actually, they are quite small; the horizontal line in the lower figure corresponds to the trend line in the upper figure.

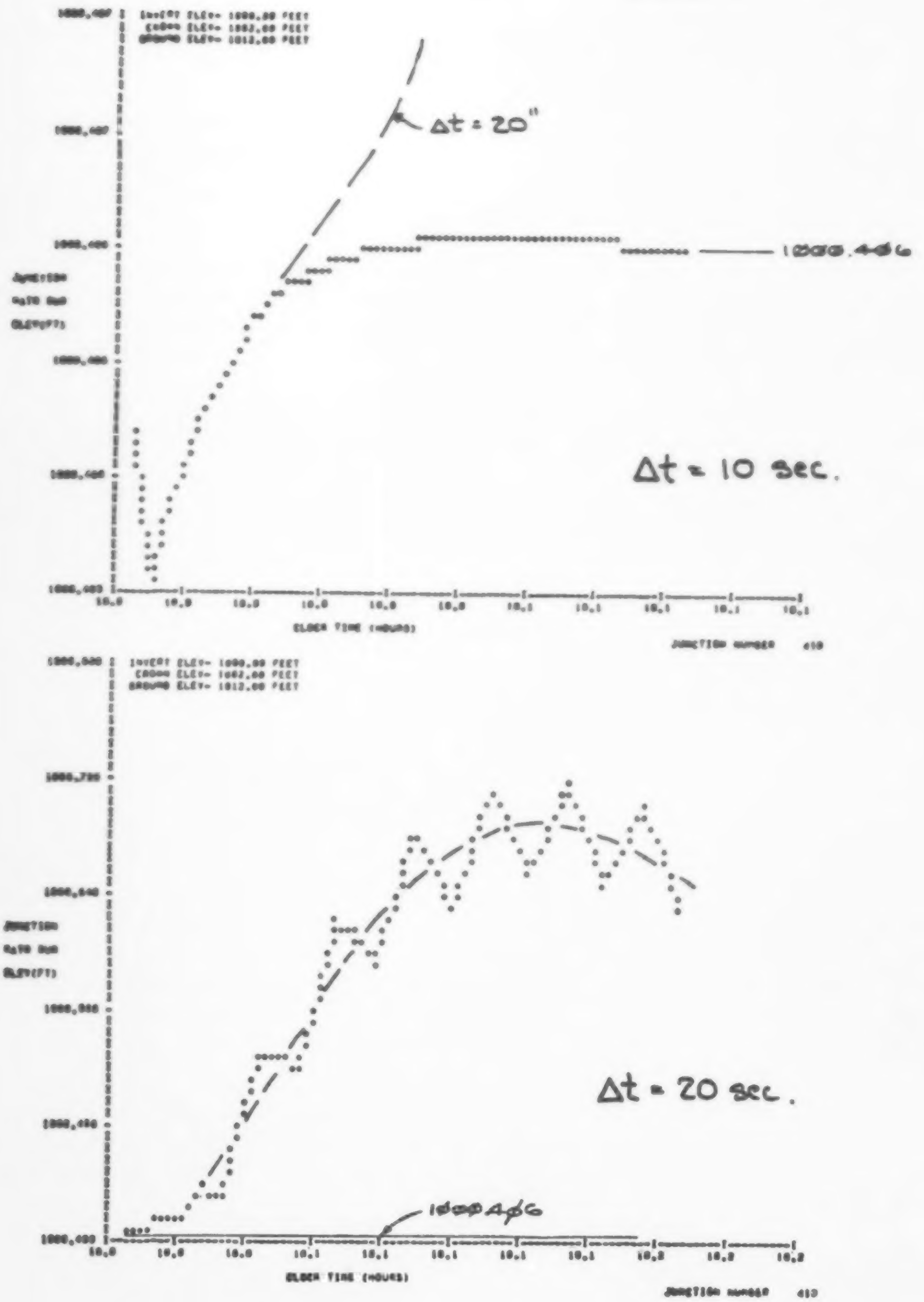

Figure 14. SIMULATED WATER LEVELS AT A MANHOLE - TEST SYSTEM

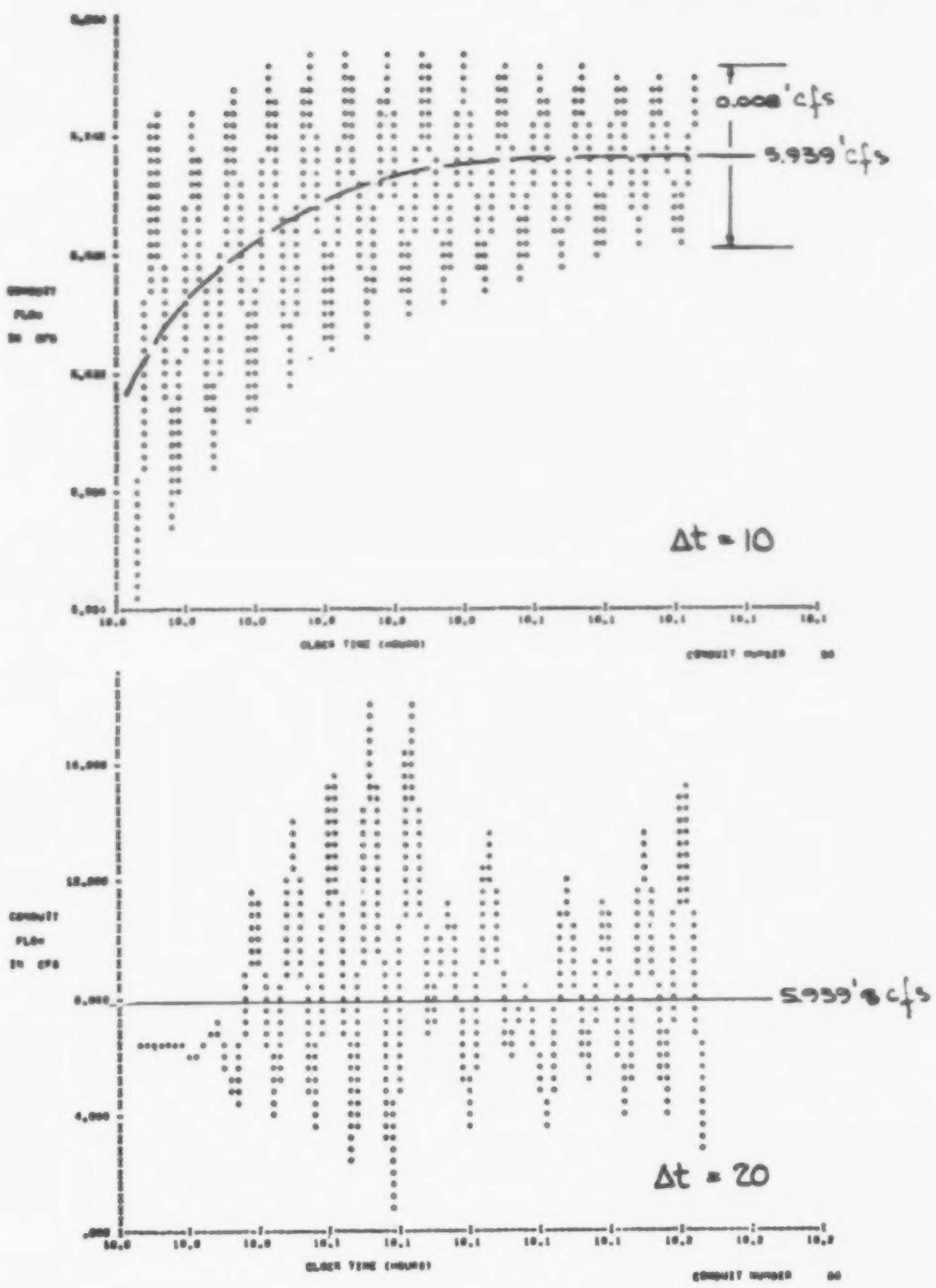

Figure 15. SIMULATED FLOWS IN A CONDUIT - TEST SYSTEM

SUMMARY AND CONCLUSIONS

Application of Storm Water Management Models is somewhat more an art than a science. There is considerable subjectivity involved and sound hydrologic sense is essential to obtaining reasonable performance. It is perhaps the general rule that there are no general cases, only specific ones. Each requires particular attention.

Despite these broad reservations there are some fairly good models; the EPA-SWMM seems to be one of the best. Examples we have chosen to review here seem to indicate that with a little patience and this model, one can obtain credible performance, that is; the model will simulate most prototype events with reasonable reliability.

There seems to be no great advantage in carrying calibration and verification to extremes, however, since the models have imbedded in them many grand assumptions. If we are aware what these are and how they may influence the model's performance, then we are in fair shape to proceed with application.

As a final caveat let us remind ourselves--and others--that after all, the EPA-SWMM is only a model; it is not the prototype. Let's not expect it to do things it wasn't designed to do. The process of improving its capabilities still goes on. After all, we modelers need to have some new challenges ahead of us.

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5. Watkins, L.H., "The Design of Urban Sewers Systems," Dept. of Sci. and Ind. Res., Road Research Laboratory, Tech. Paper No. 55, 1962.

6. Dept. of Civil Engineering, University of Cincinnati, "Urban Runoff Characteristics," Water Pollution Control Research Series, EPA, 11024 DQU 10/70, 1970.

Planning a Study Using Stormwater Management Models

By

Gerald T. Orlob*

INTRODUCTION

The great utility of the mathematical model as a tool for investigation lies in the facility with which one may expeditiously examine large numbers of complex alternatives. It may serve for critique of designs, sizing system elements, study of alternative routings, evaluating the impact of off-line storage, testing control measures, and appraising land and water use planning schemes. Given that the model can once be made to represent prototype behavior with reasonable reliability, then the user can apply it repetitively to a wide range of such alternative plans or suggested solutions to specific stormwater problems.

If the model package is well conceived and tested and the user follows the user's manual (if it exists!) explicitly, he may anticipate a productive effort with only the "normal glitches" of computer usage to plague him. However, things seldom run smoothly in the world of the computer and the math model. If something can go wrong, it will; and we have sufficient evidence to prove it.**

The purpose of this brief discussion on planning a study using mathematical models is to highlight both the capabilities of models and their fallibilities. We would like to begin by presenting a brief description of the model package, basically the Stormwater Management Model developed under the auspices of EPA. In the course of this preliminary discussion we'll try to point out where we may expect trouble and how to avoid it, if possible. We shall see that in the face of certain problems we may have to choose between the lesser of two evils, i.e., to make do with a model of limited capability or to develop a new one. Either way we must accept less than we would like to have. In the one case, we may get a rough, cheap solution and in the other, a more elegant but costly one. Which to choose?

We shall also look in some detail at the planning process itself, that is, the technique for bringing the models up to speed and to bear on the real problems to be solved. We'll pick this up somewhere along about the middle of the model development process, at calibration, and carry forward to application. In this discussion we will see that the modeler's work is never really done, it goes on forever! (But, that is the way we like it--we need the challenge ahead!).

---

*Senior Partner, G. T. Orlob & Associates, Orinda, California.

**The first half of this statement is sometimes referred to as "Murphy's Law. Murphy probably never saw a computer--the machine that surely has contributed most to transforming his early maxim into "Law".

In the conclusion of this lecture we will point toward some actual case studies. More to the point, perhaps we will set the spotlight on some actual case difficulties, and what was done to solve them. These will be discussed in more detail in the subsequent lecture.

A BRIEF DESCRIPTION OF STORMWATER MANAGEMENT MODELS

We have chosen to deal with stormwater management models in general terms, rather than the specific of the EPA-SWMM. However, many of the comments offered here apply equally well to the original model package as to other versions, components, updates or so-called proprietary packages. In all cases, the context will be that of urban stormwater, although once again many comments are just as appropos of rural storm models.

The Urban Drainage System

The Urban Drainage System will be characterized by a high proportion of impervious (or nearly impervious) surfaces. Moreover, it is a system that will include man-made impervious pathways for guiding the flow of water over the surface (curbs, gutters, lined channels, paved parking areas, streets, etc.) and underground (storm, wastewater, and combined sewers). The system includes all appurtenances that guide, control, or otherwise modify either the quantity, rate of flow or quality of runoff from urban drainage such as catch basins, storage basins, inlets, manholes, sediment traps, wiers and outfall structures.

Figure 1 illustrates a somewhat simplified urban drainage system. In this case, we view the system as an assemblage of subsystems dealing with surface runoff, transport of flow and quality, and the receiving water.

Surface Runoff Subsystem

The Surface Runoff Subsystem is illustrated for our example in Figure 2 which depicts the drainage area tributary to a sewer inlet as a system of surface elements (the rectangles), gutters (the dotted lire) and drainage ditches (the dashed line). For the Surface Runoff Module of the EPA-SWMM the following data are required for each catchment:

1. landuse (5 classes)
2. total surface area, acres
3. total length of streets (gutters) 100's feet
4. percent imperviousness
5. average watershed slope
6. average street (gutter) slope
7. point in transport subsystem to which the subarea drains

Input to the subsystem is comprised of rainfall that may be described in terms of an intensity-time graph derived from direct measurements in the watershed and applied to each subarea in accordance with the storm's

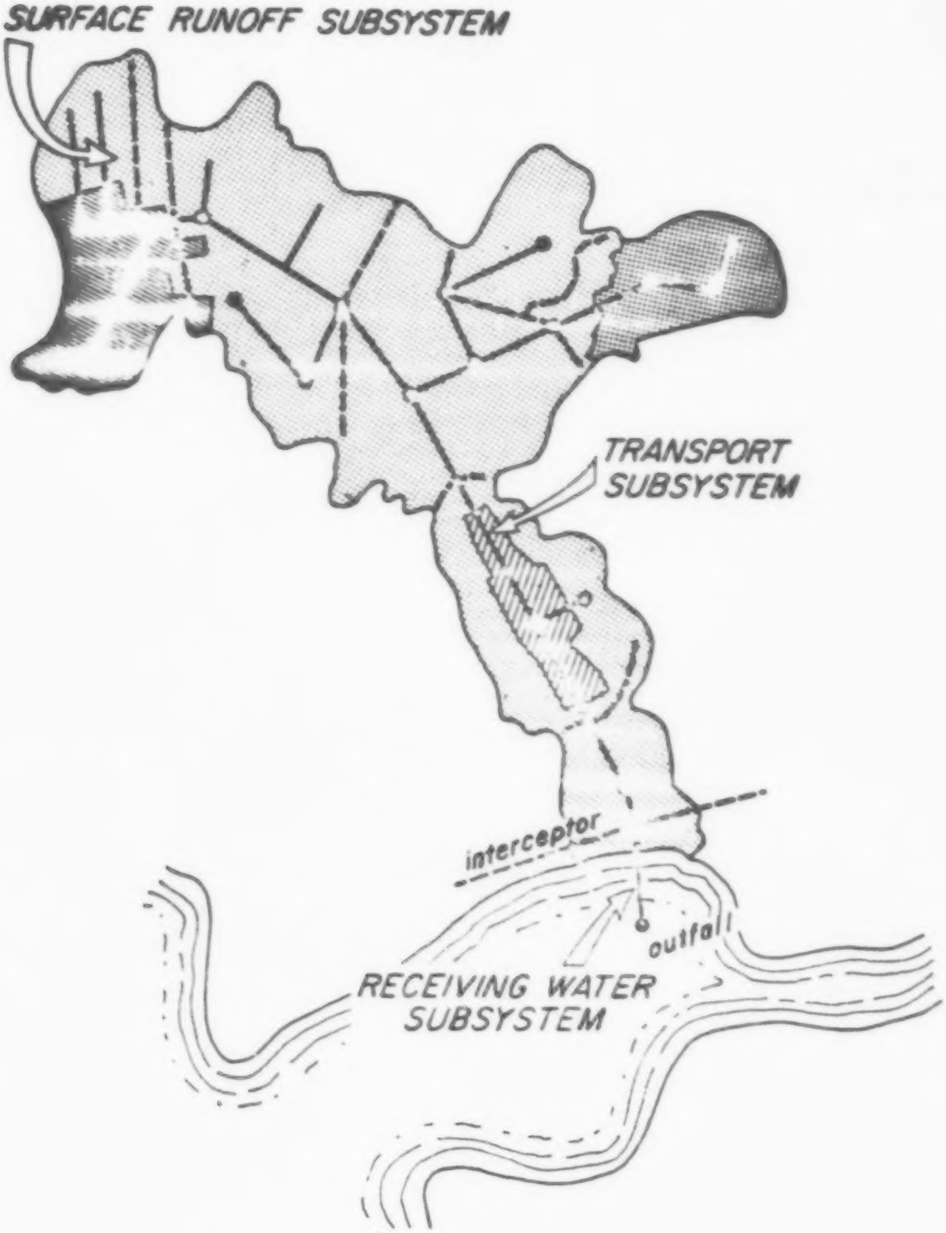
SURFACE RUNOFF SUBSYSTEM
TRANSPORT SUBSYSTEM
interceptor
outfall
RECEIVING WATER SUBSYSTEM

Figure 1. The Urban Drainage Subsystem

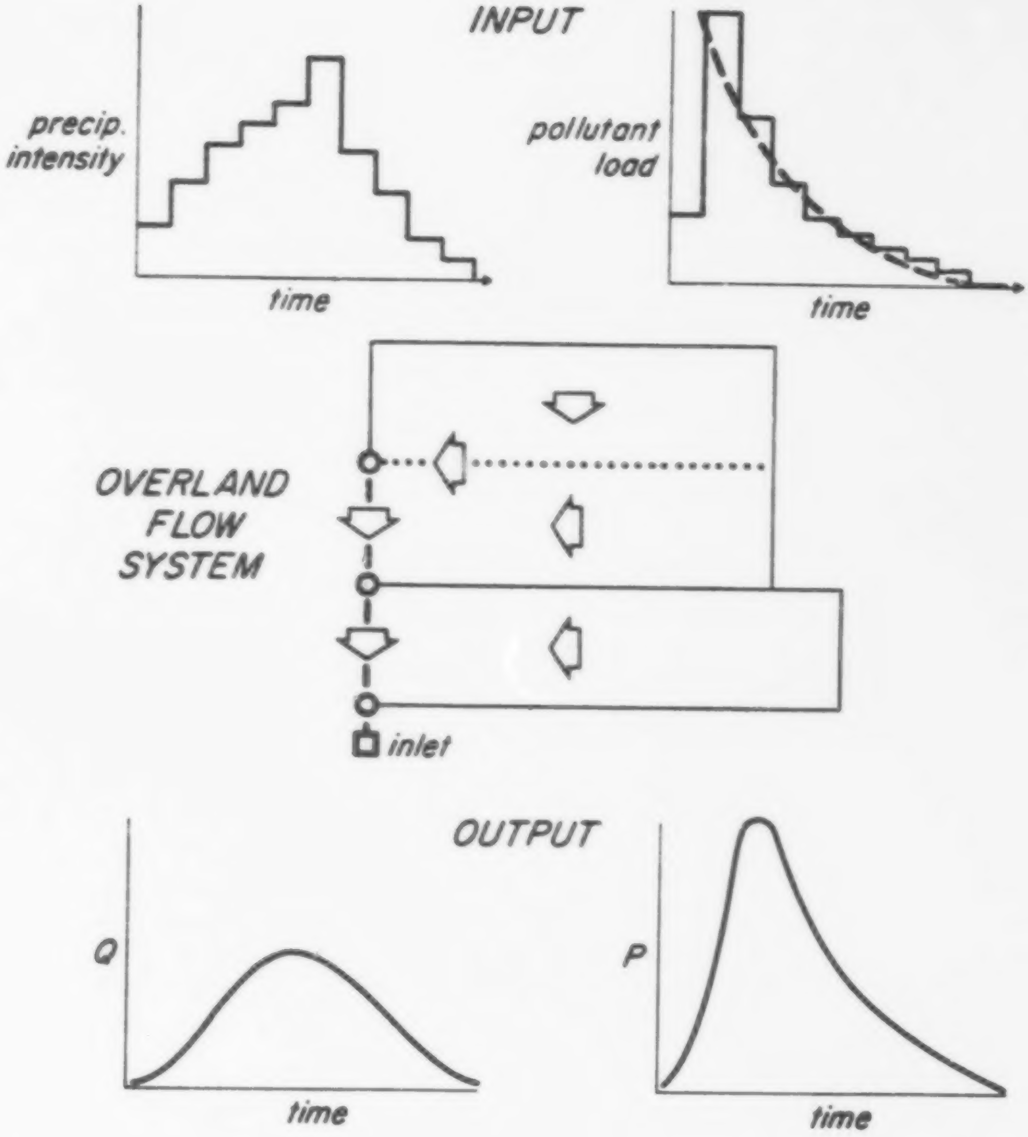
INPUT
precip. intensity
time
pollutant load
time
OVERLAND FLOW SYSTEM
inlet
OUTPUT
Q
time
P
time

Figure 2. Surface Runoff Subsystem

pattern. The inset in the upper left of Figure 2 illustrates a typical rainfall *hyetograph*. Values are given in inches per hour, usually for every five minutes of the storm.

Within the subsystem a certain mass of a quality constituent (pollutant) may exist at the outset of the storm and be delivered by the flow at mass rates and concentrations that may depend on the nature of the storm, the character of the surface, and the sources of the pollutant (upper right insert, Figure 2). Data are provided on rates of pollutant accumulation, pounds per 100 feet gutter per day (up to 17 types) and pounds of pollutant per pound of dust and dirt delivered to the watershed.

The overland flow process modifies the rainfall hyetograph by infiltration, surface retention and transient storage, so that at the inlet one observes a much modified *inlet hydrograph*, a temporal description of inlet flow (lower left insert, Figure 2). In addition, the combined flow and quality processes produce an *inlet pollutograph*, a time-concentration graph of a particular pollutant as it leaves the Surface Runoff Subsystem and enters the waste water conveyance system (lower right insert, Figure 2). These two graphs, one of flow and the other of quality, comprise the *output* of the Surface Runoff Subsystem and are *input* to the Transport Subsystem.

Transport Subsystem

The *Transport Subsystem* is comprised of the physical works for conveying stormwaters and their associated pollutant loads from all of the inlets in the system through a network of underground conduits to a point (or points) of disposal. Enroute, flow and quality are both modified by accretions to the system from other tributary areas and/or point sources of pollution. In addition, flows and pollutant concentrations are attenuated in passing through the system, the degree of modification depending on such factors as system storage, "off channel" storage, phase relationships of inflow hyetographs and pollutographs and certain hydraulic properties of the system. The two lower inserts in Figure 3 illustrate a typical set of outputs from the Transport Subsystem, a hydrograph and a pollutograph, that in turn become inputs to the receiving water.

Receiving Water Subsystem

The *Receiving Water Subsystem* may be a stream, a lake, an estuary or a coast. Discharge into an estuary will be used for illustration.

The impact of the discharge on the estuary will probably be assessed in terms of the concentration of a particular quality constituent: its distribution in space, its persistence in time, and its frequency of exceedance of a certain critical level. For a given hydrological event, the system may be observed synoptically (at the same instant in time) or temporally (at the same point in space).* One gives the distribution in space, the other the persistence in time. From the standpoint of quality

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*These are sometimes referred to as the Lagrangran and Eulerian viewpoints, respectively.

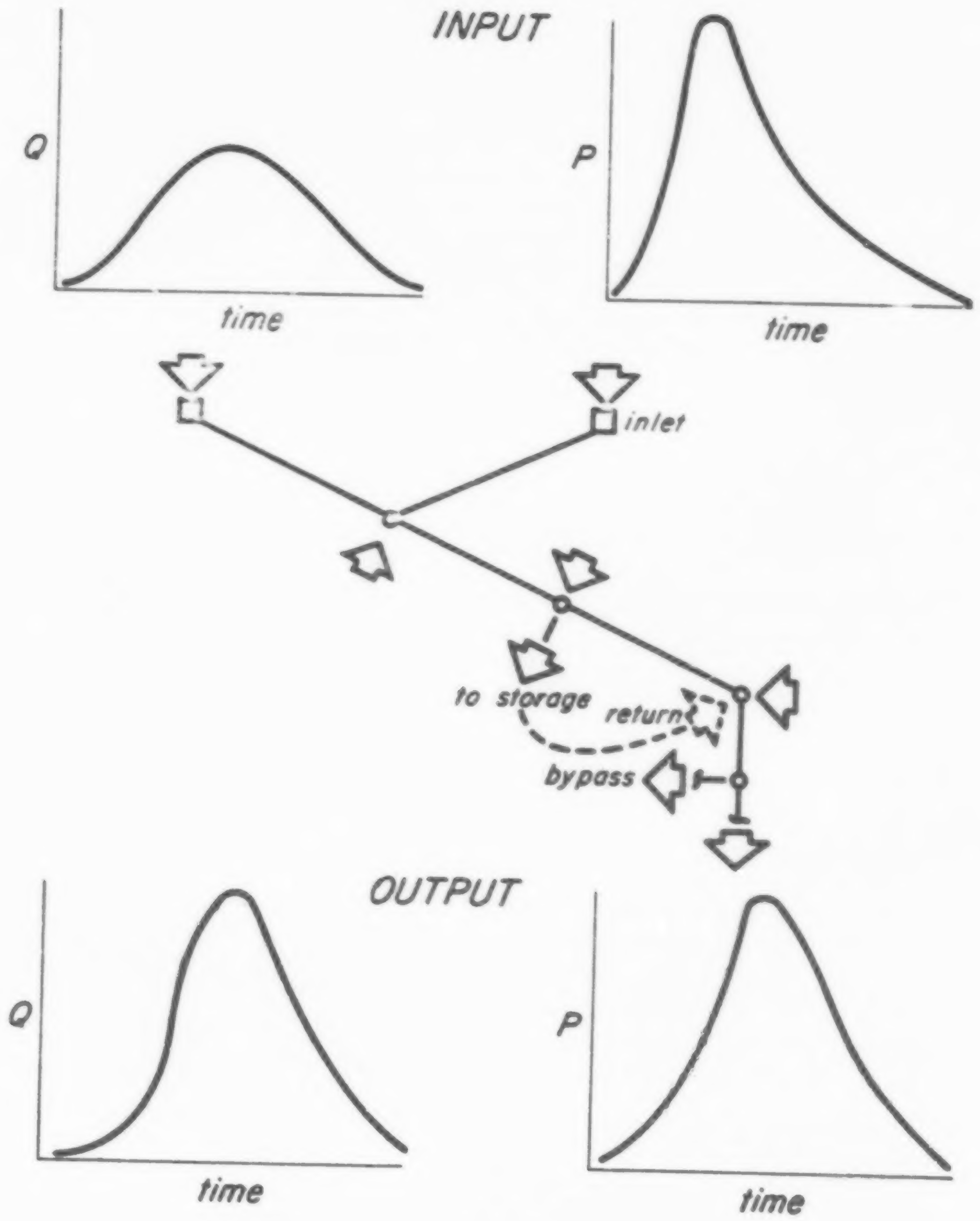
INPUT
Q
time
P
time
inlet
to storage
return
bypass
OUTPUT
Q
time
P
time

Figure 3. Transport Subsystem

management both viewpoints are usually required for each hydrological event. To obtain frequency of exceedance of a critical level the impact on the receiving water must be observed a "statistically significant" number of times. Figure 4 illustrates typical responses ("impacts") for an estuary.

TECHNICAL APPROACH TO MODEL APPLICATION

Application of models in real problem solving involves some necessary preliminary steps that really begin with the conceptualization of the model itself. For the purposes of our discussion here, however, we may assume that we already have a tangible operating model in hand and that the preliminaries of conceptualization, formulation, and computational representation as outlined schematically in Figure 5 have already been completed. We will pick up the process at this point and describe briefly what is entailed in each of the steps of calibration, verification, documentation, and application.

Calibration

Calibration entails adjusting empirical relationships in the model to ensure replication of *in situ* observation of the state variables under known boundary conditions. This exercise is usually performed for a case considered representative of those most likely to be explored in model application.

The process is primarily subjective, involving adjustment by the modeler of empirical coefficients during a succession of applications of the model until the model response is in agreement with prototype data. The degree of conformity between model and prototype that constitutes "agreement" is generally judgmental, since many factors, some outside of the modeler's control, influence results. Experience plays an important role here.

A related part of this effort involves *sensitivity testing* of the model, i.e., evaluating the model's response in terms of selected state variable to changes in coefficients, boundary conditions, input data, and sometimes even the structure of the model itself. A product of calibration-sensitivity testing is a model suitable for application as a predictive or management tool.

Verification

Verification is the modeler's demonstration of the model's predictive capability--the first attempt to apply the model to a new set of conditions and to generate responses that are representative of the prototype. Taking the model as calibrated to known data, the modeler attempts to simulate an unknown (to him) system response. The test of predictive capability is whether the model can produce results that are within the range of acceptability used in calibration.

Sometimes in this step a recalibration is indicated; then, the processes of calibration and verification are repeated until the results are acceptable. A final product is a model ready for documentation.

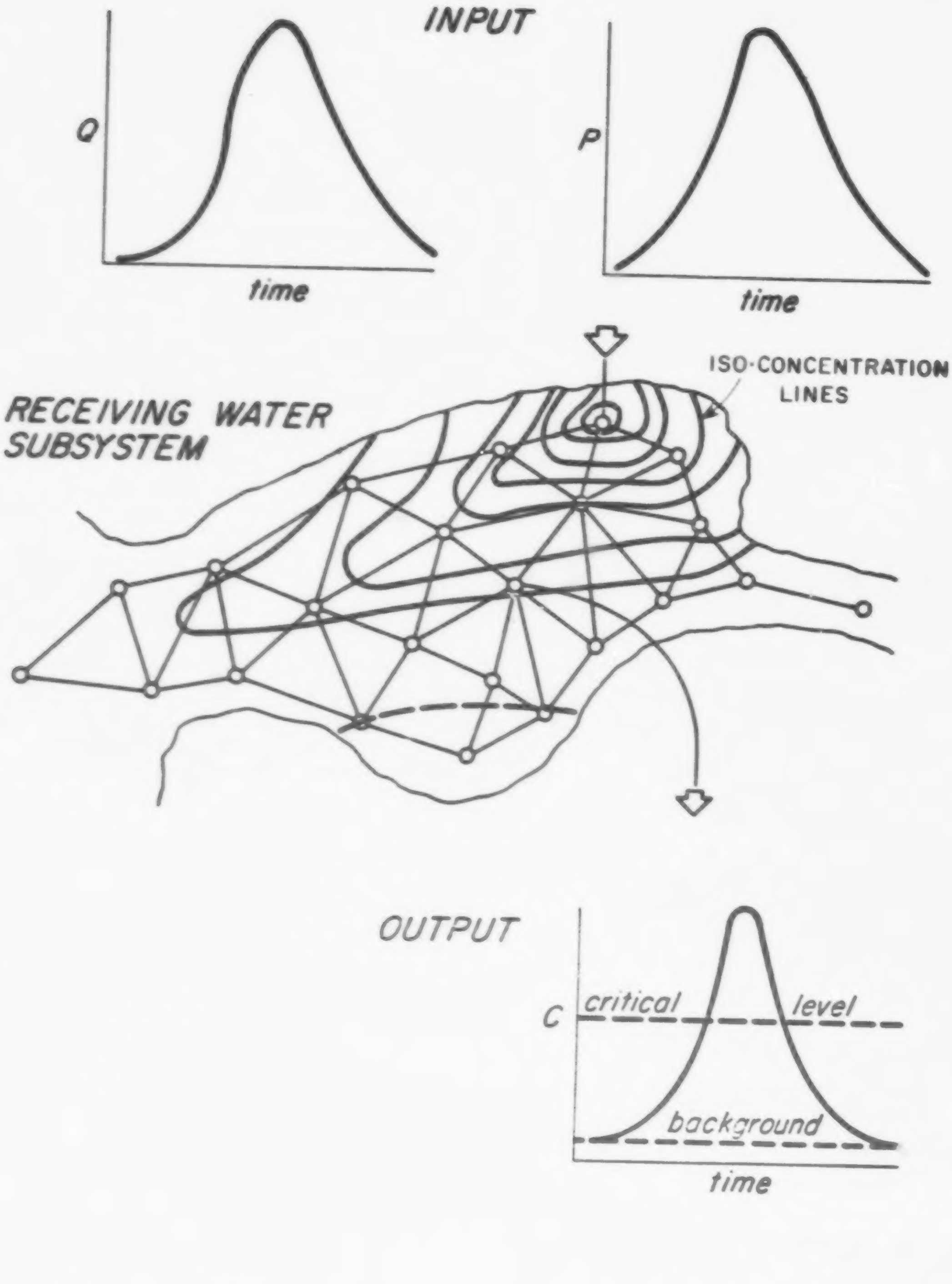
INPUT
Q
time
P
time
ISO-CONCENTRATION LINES
RECEIVING WATER SUBSYSTEM
OUTPUT
C
critical
level
background
time

Figure 4. Receiving Water Subsystem

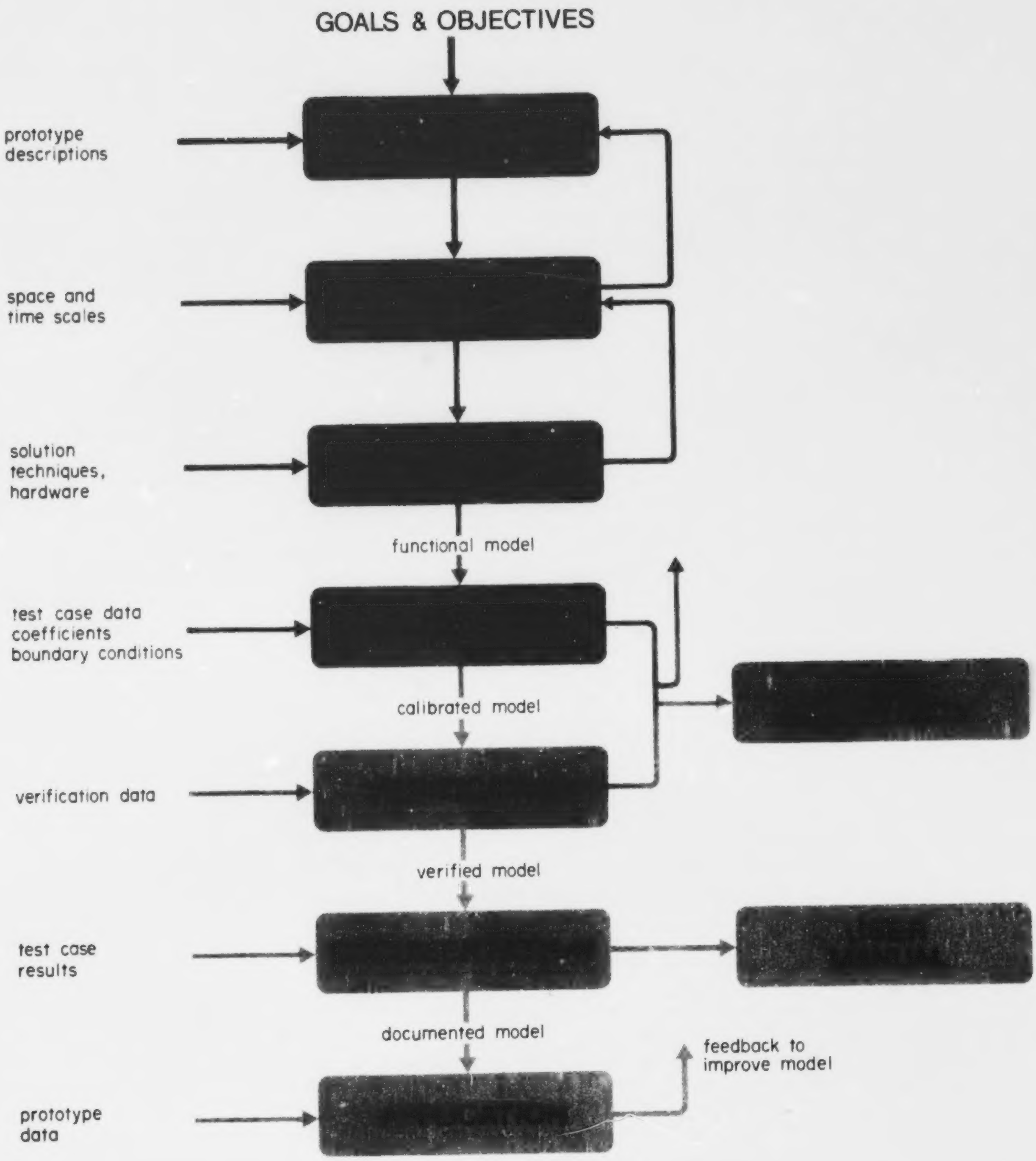
GOALS & OBJECTIVES
prototype descriptions
space and time scales
solution techniques, hardware
functional model
test case data coefficients boundary conditions
calibrated model
verification data
verified model
test case results
documented model
feedback to improve model
prototype data

Figure 5. The Processes of Mathematical Model Development Conceptualization to Application

Documentation

Documentation entails complete description of the model, its conceptual foundation, formulation, computational representation, data formats, input-output requirements and options, and example problems with solutions. Usually, the program is documented separately in detail, providing flow diagrams, variable descriptions, characteristic dimensions and a program listing and is accompanied by a Users' Manual that describes how to apply it to a new case and gives a demonstration example. This has been accomplished for the EPA-SWMM under the original development contracts (1). Under a continuing program of new development, improvements in the model will also be documented and the Users' Manual revised from time to time.

Application

Application is a continuing process in model development that assures gradual improvement through actual usage. No model is infallible; when difficulties are encountered there is no assurance that the model, itself, is infallible. After all, it is only a "model," not the prototype itself, and should always be subject to reexamination and possible improvement.

Thus, the model development process is not closed, but rather it is open-ended and requires that the user and model developer continually reexamine the model, the problems it is being asked to solve, and the data provided. This should result in progressive improvement in capability to simulate the prototype's behavior, commensurate with practical need and available resources.

SOME COMMENTS ON DATA

Calibration, verification, and application require that reliable information be obtained from the prototype. Often data on actual storm events are difficult to obtain or may not exist at all for the drainage in question. Invariably, the raingage lies outside the actual drainage basin, even miles away. We must somehow relate precipitation data for this gage to gaged outflow from the basin. The usual hydrologic techniques of inter-correlation between stations or construction of isohyetal patterns over the larger area that includes our basin may be used to develop the necessary rainfall-runoff relationship.

In this process, attention must be given to the temporal detail of precipitation data. If the basin is small, say under 100 acres or so, then it will not suffice to have hourly readings; we will need data at 5-minute spacings, surely not less than 10-minute intervals. If data are too coarse we will lose definition of the storm and may not be able to calibrate the model closely enough to allow description of runoff peaks.

Measurement of the flow itself presents many difficulties. Of course, what we measure will be the net contribution to surface runoff after satisfying interception, depression storage, and infiltration. Since we have no direct way of measuring these other quantities we must distribute the aggregate difference between our uncertain input (precipitation) and our measured output arbitrarily to each. Fortunately, the functional relationships are distinctive; interception and depression storage may be considered as constant amounts of precipitation while infiltration varies throughout the storm in some unique fashion, e.g. exponentially decreasing from some initial value. These assumptions (that may not always hold) help us to make some reasonable judgments about the amount of the flow that is not measured as runoff and how it is distributed temporally. The adjustment process, that of reconciling measured rainfall and gaged runoff, is basically subjective. There is really no substitute here for a basic understanding of the hydrologic phenomena and experience in their interpretation.

ROLE OF MODELS IN DECISION-MAKING

A model may be looked upon as an *adjunct to decision-making*, a tool that when wisely applied in the assessment of alternative strategies can enhance the decision makers' own judgment. It is really a representation of the real world that permits the user to test his ideas, his preferences or some conditions of prototype behavior before the fact of change. He can make observations at low cost, quickly, and without permanent alteration of the real world system. He can, with the aid of a properly conceived model, explore interactions of such complexity that intuitive assessment is impossible.

This is very much the case for the Storm Water Management Models. Once the models are calibrated and verified, providing the user with some appreciation of their reliabilities, and fallibilities, the models can be employed best to discriminate between alternative choices. For example, he may wish to test the effect of changing a land use pattern on the runoff peak and time of occurrence. Intuitively, he would expect that if he adds more impervious surfaces, the peak will rise; yet he may not be able to say much about the magnitude of the peak and when it will occur, except with the aid of the model or its equivalent. He can also test the effect of the areal distribution of the added increment of impervious surface. Where should it be placed to produce the least impact on the basin's hydrologic response? Such questions can be answered in an organized program of model application.

If we may assume that we have in hand a reasonably good model, we can now turn to a closer examination of some of the problems in applying it. In the next lecture we will examine some actual case studies where such a model has been employed in prediction of urban runoff events and where certain common problems have been encountered and solved.

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THREE CASE STUDIES ON THE APPLICATION OF THE STORM WATER MANAGEMENT MODEL

By

Jekabs P. Vittands*

CASE STUDY - 1 SEWER SYSTEM MODELING AND IMPROVEMENTS IN THE EASTERLY DISTRICT, CLEVELAND, OHIO

Introduction

The Easterly District of the City, shown on Figure 1, totals approximately 16,000 acres and includes the downtown area, with an additional 25,000 acres tributary from surrounding areas outside the City.

The existing sewerage system within the Easterly District is almost entirely combined. Tributary areas outside of the City use City sewers and drains for conveyance of drainage to downstream water courses.

Flooding problems due to inadequate sewer capacity were a major concern of the project. During the eight months from January 1 to August 31 in 1972, over 560 complaints were registered for cellar flooding, while another 168 complaints were registered for pipe cave-ins. These pipe failures, or cave-ins, are due to the existence of a large proportion of old brick sewers, many of which are over seventy years old. The locations of these flooding and cave-in complaints are shown on Figure 2.

Because of the inadequate capacity and condition of many of the sewers, a program was started for the systematic evaluation of the existing system, and preparation of a plan and program for the reconstruction of the inadequate segments of the sewer system. This project was the first step towards solving flooding problems in the Easterly District of the City, with detailed design and construction as the next steps.

The specific purposes of the project were to determine the improvements needed to eliminate flooding problems via computer modeling of the sewer system in the Easterly

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*Project Manager, Metcalf & Eddy Engineers, Boston, Massachusetts.

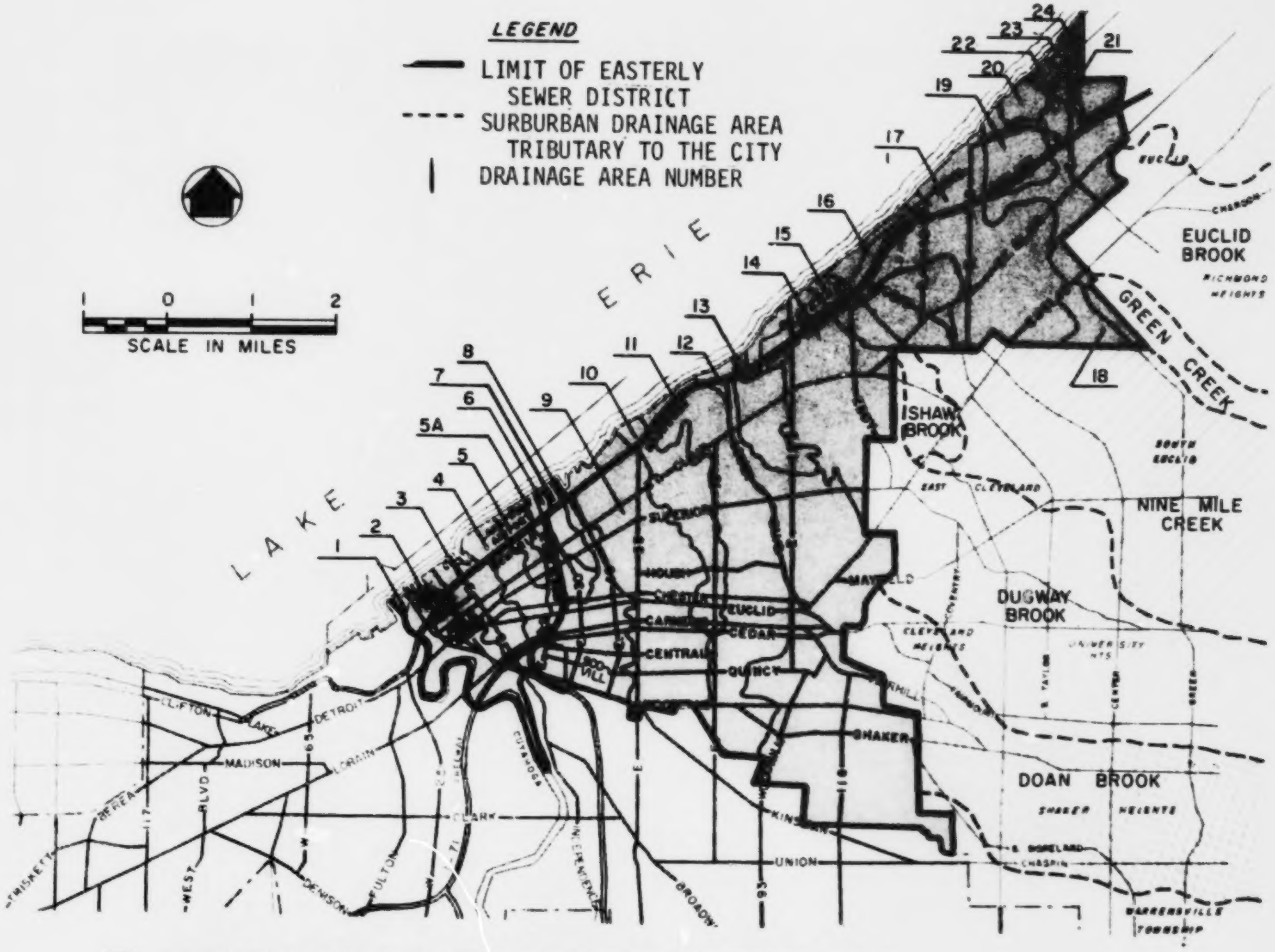
LEGEND
LIMIT OF EASTERLY SEWER DISTRICT
SURBURBAN DRAINAGE AREA TRIBUTARY TO THE CITY
DRAINAGE AREA NUMBER
SCALE IN MILES
1
0
1
2
LAKE ERIE
1
2
3
4
5
5A
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
EUCLID BROOK
RICHMOND HEIGHTS
GREEN CREEK
ISHAW BROOK
SOUTH EUCLID
EAST CLEVELAND
NINE MILE CREEK
DUGWAY BROOK
CLEVELAND HEIGHTS
UNIVERSITY HTS
DOAN BROOK
SHAKER HEIGHTS
WARRENSVILLE TOWNSHIP
SUPERIOR
HOUGH
CHESTER
EUCLID
CARNEGIE
CEDAR
CENTRAL
QUINCY
MAYFIELD
SHAKER
KINSMAN
UNION
CLARK
CUYAHOGA
INDEPENDENCE
BROADWAY
DETROIT
LAKE
CLIFTON
MADISON
LORAIN
BEREA
WEST BLVD
FULTON
DENISON
TRISKETT
CHAGRIN
E. MORELAND
TAYLOR
CENTER
GREEN
117
65
55
105
93
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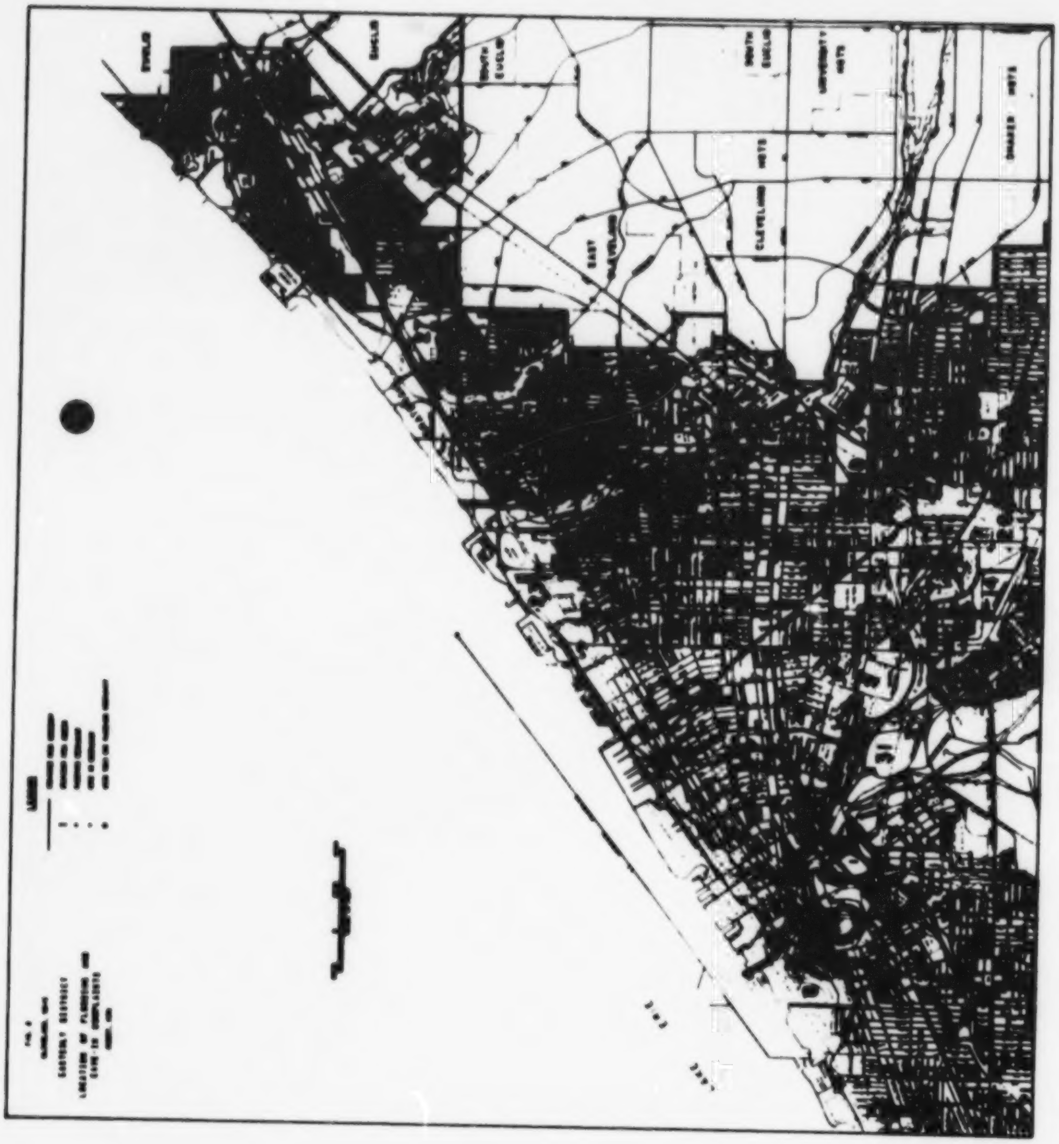
FIG. 1 DRAINAGE AREAS, EASTERLY DISTRICT, CLEVELAND, OHIO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EASTERLY DISTRICT
LOCATION OF FLOODING AND
CAVE-IN COMPLAINTS

FIG. 2 LOCATION OF FLOODING AND CAVE-IN COMPLAINTS – CLEVELAND, OHIO

District of the City, and to identify orders of priority for necessary improvements along with estimated costs. Another purpose of this project was to install this modeling capability at the City by providing computer programs and training City personnel in the use of the model.

Recognizing that in combined sewer systems the elimination of flooding conditions is only one part of the problem and that, in addition, remedies for overflow pollution will have to be determined in the future, consideration was given to this need in the modeling of areas using the Storm Water Management Model (SWMM). Only the parameters necessary for the quantification of pollutants will have to be added to the data in order to use the Storm Water Management Model as an aid in determining alternative remedies for combined sewer overflows.

The approach to the project was to use the applicable computer programs of the SWMM to carry out studies for necessary improvements. The two major blocks of the programs used were the RUNOFF Block and the TRANSPORT Block which were modified to permit their more effective use in this project. The EXECUTIVE Block was used in addition to provide its function of interprogram coordination.

The basic technique used was one of design by analysis wherein computer simulation is used to determine the conditions in a sewer system during a storm. An actual or selected design rainfall is assigned to a drainage area with a specific sewer configuration and the flow in sewers and flooding conditions are determined by simulation.

In this project, first, the flooding conditions and transport capabilities in the existing system were determined by computer simulation. Then, based on these results, the necessary sewer sizes were estimated to eliminate flooding under the design storm conditions. Computer simulation was then used to test the adequacy of the selected improvements. In some cases, a number of trials were necessary in order to arrive at the solution.

All sewers 30 inches in diameter (or equivalent in size) and larger were modeled explicitly. In addition, in certain areas where the nature of the tributary area is complex, smaller pipes were also included. These models were referred to as large models and an example is shown on Figure 3 for Drainage Area No. 3.

Pipes smaller than 30 inches in diameter were analyzed on a tributary area basis. These were referred to as fine models. In this case, selected typical areas

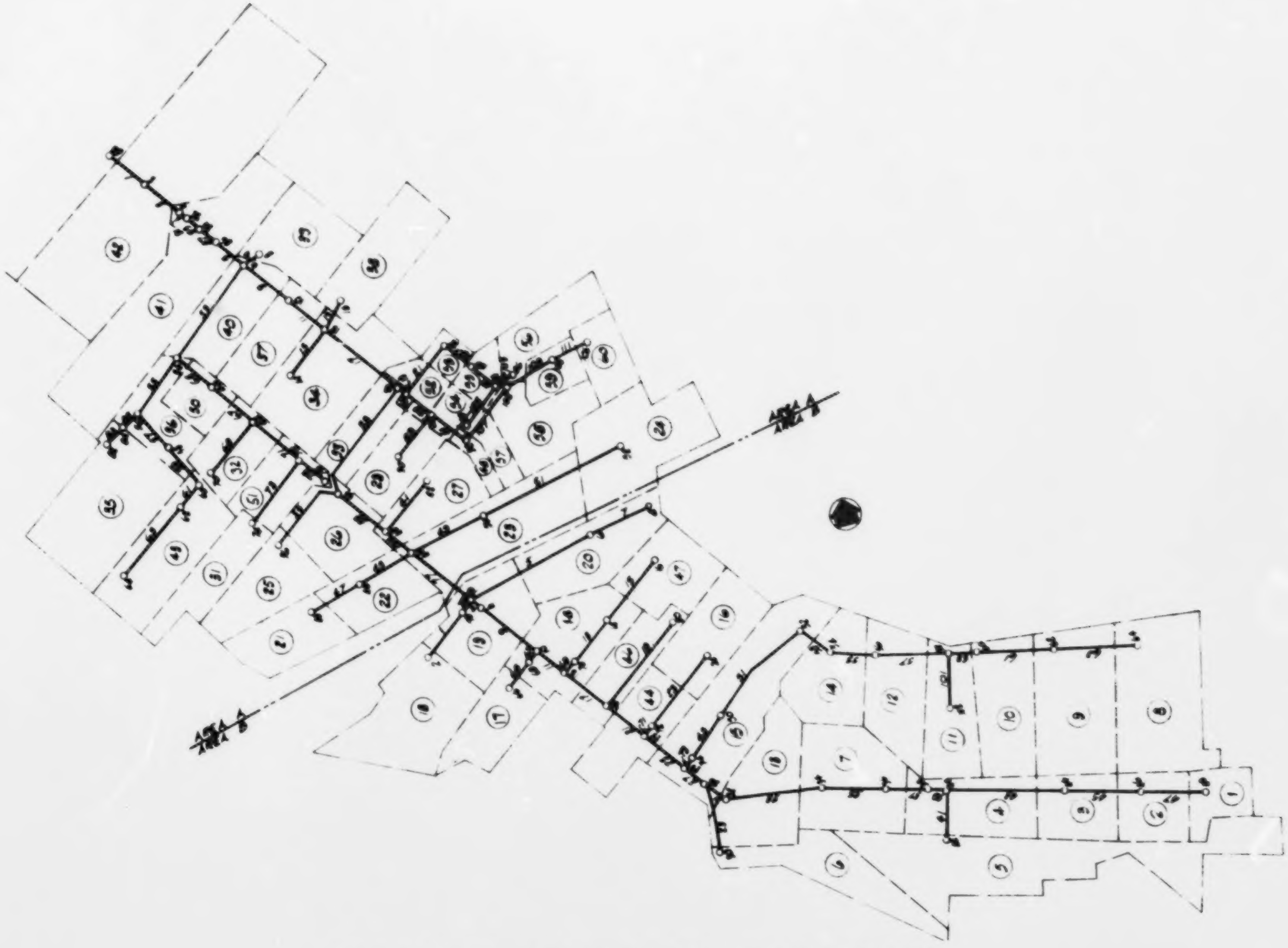


FIG. 3 LARGE MODEL SUBDIVISION – CLEVELAND, OHIO

were modeled in order to generate tables for flow quantification which could be used to determine the required size of sewers in the area on the basis of tributary area characteristics such as size, slope, and percent imperviousness. This approach is discussed further under the section entitled Small Model Development.

Design Criteria

This section discusses only design criteria of the project pertinent to the use of the SWMM.

The criteria established by the City for the degree of protection to be provided was as follows:

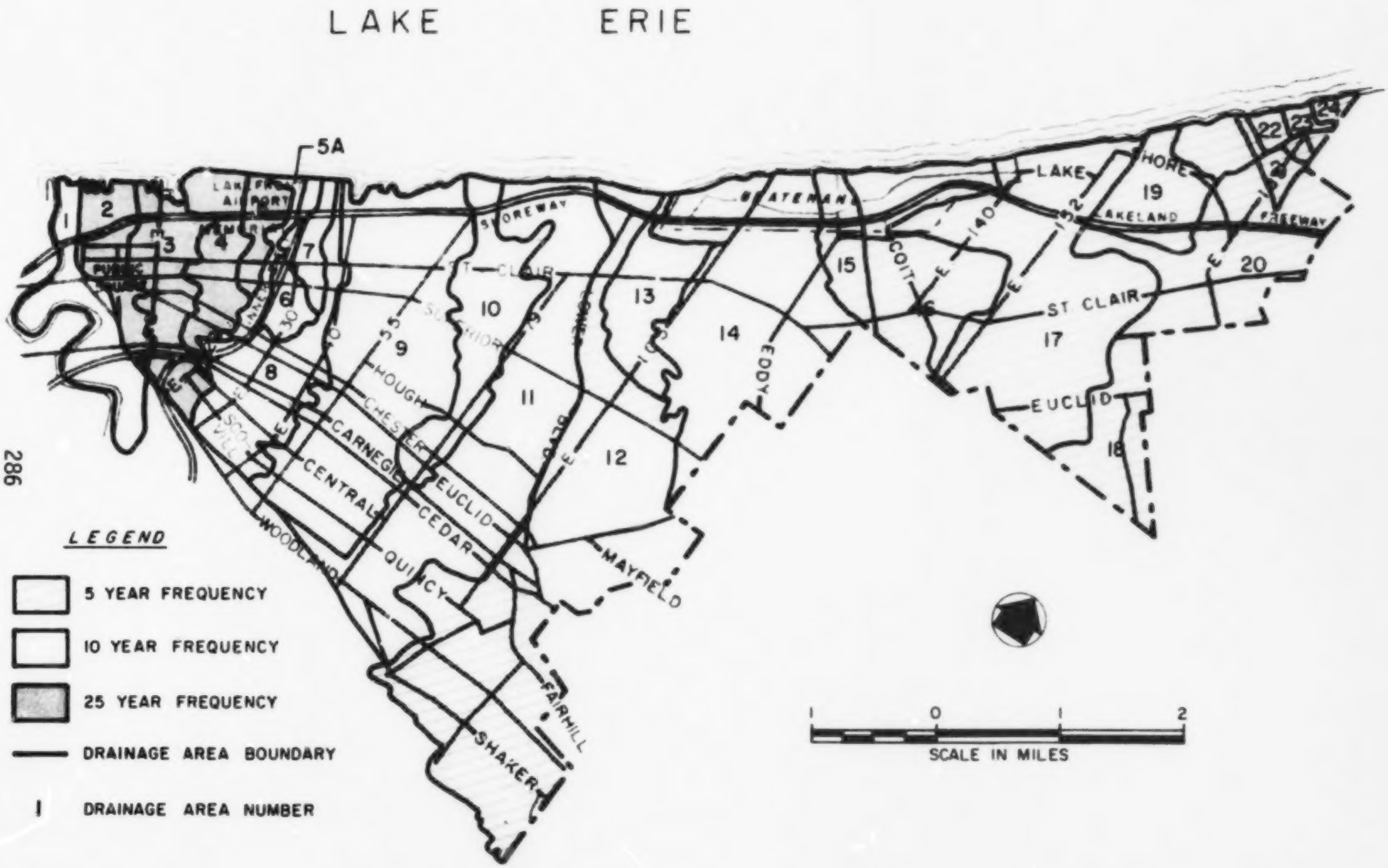
1. Downtown areas - 25-year storm
2. Industrial and commercial areas - 10-year storm
3. Residential areas - 5-year storm

The design storms used in each of the drainage areas are shown on Figure 4.

Hyetograph Development. In using simulation as a technique for flow determination, time-varying parameters are required. On this basis, rainfall data representative of actual conditions are necessary, rather than using time-averaged rainfall values which are then multiplied by a constant to obtain peak runoff as is done in the Rational Method.

The procedure used in developing typical rainfalls for the above three design storms is as follows:

1. Select the desired range of frequencies and durations for the storms, i.e. 5-, 10-, and 25-year storms and 1- and 6-hour durations. Durations of 1 and 6 hours were initially judged to represent the range of storm durations applicable to the drainage area sizes in the Easterly District.
2. Using the US Weather Bureau Technical Paper No. 40, select the appropriate isopluvial maps corresponding to the particular storm frequencies and durations. Locate the study area on the maps and record the rainfall depths, interpolating between isopluvial lines. This rainfall depth information is presented in Table 1. Also shown are the average rainfall intensities corrosponding to the depths and durations.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SUPERIOR
HOUGH
CHESTER
CARNEGIE
EUCLID
CEDAR
CENTRAL
QUINCY
WOODLAND
MAYFIELD
FAIRHILL
SHAKER
EDDY
BRATENAHL
COIT
LAKE
SHORE
LAKELAND
FREEWAY
E 30
E 55
E 79
E 105
E 140
E 152
LEGEND
5 YEAR FREQUENCY
10 YEAR FREQUENCY
25 YEAR FREQUENCY
DRAINAGE AREA BOUNDARY
DRAINAGE AREA NUMBER
1 0 1 2
SCALE IN MILES

FIG. 4 DESIGN STORM FREQUENCIES, EASTERLY DISTRICT, CLEVELAND, OHIO

For rainfall durations less than 30 minutes, isopluvial maps do not exist. However, Technical Paper No. 40 provides constants for converting depths at 30-minute durations to depths at 5-, 10-, and 15-minute durations.

3. Select the desired time increments for each storm duration. A 5-minute increment was chosen for the 1-hour storm and a 10-minute increment was chosen for the 6-hour storm. This was judged to best describe the storm characteristics and not consume large amounts of computer time.

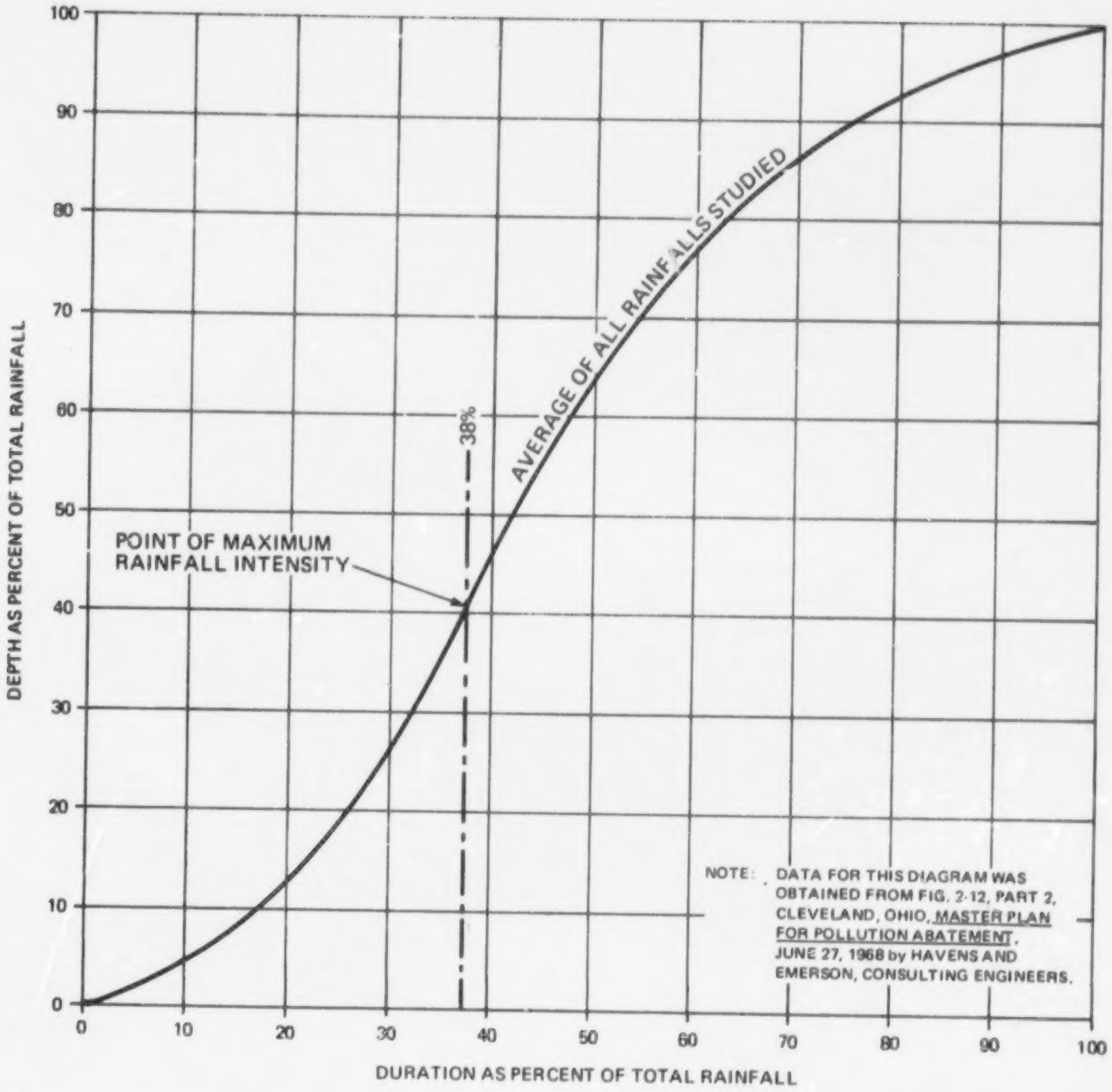
4. Select a storm pattern. Special consideration must be given to locating the rainfall peak at a point from beginning of storm that is representative of storms in the Cleveland area. Figure 5 was used as a basis for this. Figure 5 shows the average distribution of rainfall and the time of maximum rainfall intensity from beginning of storm.

5. The design hyetographs for a storm of a given frequency and duration were prepared as follows:

   a. A bar graph was drawn with TIME in minutes plotted on the x-axis and RAINFALL DEPTH in inches plotted on the y-axis.

   b. The point in time was located where the maximum rainfall intensity will occur. For a 1-hour storm duration, this point occurs 23 minutes after the start of the storm according to Figure 5.

   c. At the point of maximum intensity, the rainfall depth corresponding to the selected time increment was plotted. For a 1-hour storm with a 5-minute time increment, the rainfall depth is 0.44 in., while the rainfall depth for a 10-year frequency storm is 0.49 in.



NOTE: DATA FOR THIS DIAGRAM WAS OBTAINED FROM FIG. 2-12, PART 2, CLEVELAND, OHIO, MASTER PLAN FOR POLLUTION ABATEMENT, JUNE 27, 1968 by HAVENS AND EMERSON, CONSULTING ENGINEERS.

FIG. 5 CLEVELAND, OHIO, RAINFALL MASS DIAGRAM

TABLE 1. RAINFALL DEPTH AND INTENSITY - DURATION - FREQUENCY RELATIONSHIPS, CLEVELAND, OHIO

	Rainfall depths in inches and intensities in inches per hour for various frequencies							
	1 year		5 years		10 years		25 years	
Duration	in.	in./hr	in.	in./hr	in.	in./hr	in.	in./hr
5 min.	.28	3.36	.44	5.28	.49	5.88	.58	6.96
10 min.	.43	2.58	.68	4.07	.76	4.55	.90	5.40
15 min.	.54	2.16	.86	3.44	.96	3.84	1.14	4.56
30 min.	.75	1.50	1.19	2.38	1.34	2.68	1.58	3.16
1 hr.	.95	0.95	1.50	1.50	1.70	1.70	2.0	2.0
2 hr.	1.13	0.56	1.65	0.83	2.10	1.05	2.2	1.1
6 hr.	1.50	0.25	2.3	0.38	2.65	0.44	2.95	0.49

d. Using the location of the hyetograph peak as a starting point, rainfall depths were plotted for each time increment corresponding to the storm pattern indicated on Figure 5. The rainfall depth for any given time period should not exceed the maximum depth indicated in Table 1 for a similar period.

e. The ordinate of the above graph should then be converted to rainfall intensity in inches per hour to produce the design hyetographs.

Figure 6 shows the design hyetographs for 5, 10, and 25-year storms of a 1-hour duration. It should be noted that these hyetographs do not represent actual storm events, but synthetic storms fulfilling the U. S. Weather Bureau statistics on rainfall depth-duration-frequency and the average rainfall distribution pattern in the Cleveland area.

Similar hyetographs were developed for storms of a 6-hour duration. Both the 1-hour and 6-hour duration storms were tested against a selected large drainage area in the Easterly District. Area No. 8 was used to determine improvements necessary under both storms. Area No. 8 was selected on the basis of its size. From this it was found that there were no marked differences in the results allowing selection

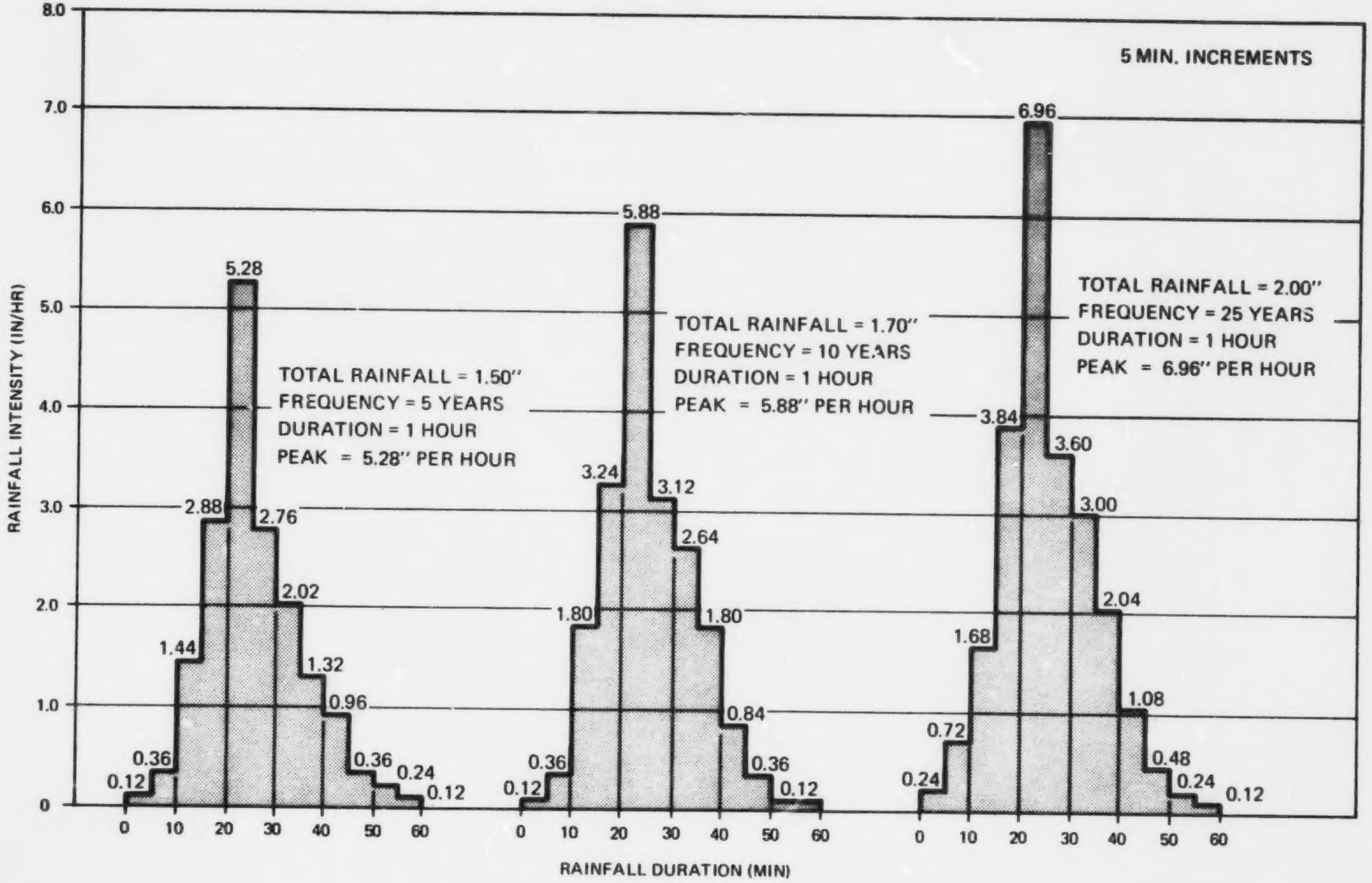


FIG. 6 5, 10 and 25-YEAR 1-HOUR DESIGN STORMS, CLEVELAND, OHIO of the 1-hour duration storms as a basis for design. Figure 6, therefore, represents the design storms used in this project.

Land Use and Developments. For design of facilities for urban runoff, it is necessary to measure the degree of development in terms of the extent of roofs and pavements, and other impervious surfaces in an area tributary to a sewer. The data used in this report is shown on Figure 7.

Hydraulic Parameters. The following criteria were used in the Storm Water Management Model:

Surface retention storage

1. impervious areas: 1/16 in. (inches)
2. pervious area: 1/4 in.

Ground infiltration as used in Horton's Equation

1. maximum rate: 3.0 in/hr. (inches per hour)
2. minimum rate: 0.52 in/hr.
3. decay rate: 0.00115 in/sec.

Flow friction factors as used in Manning's Equation

A. Overland flow

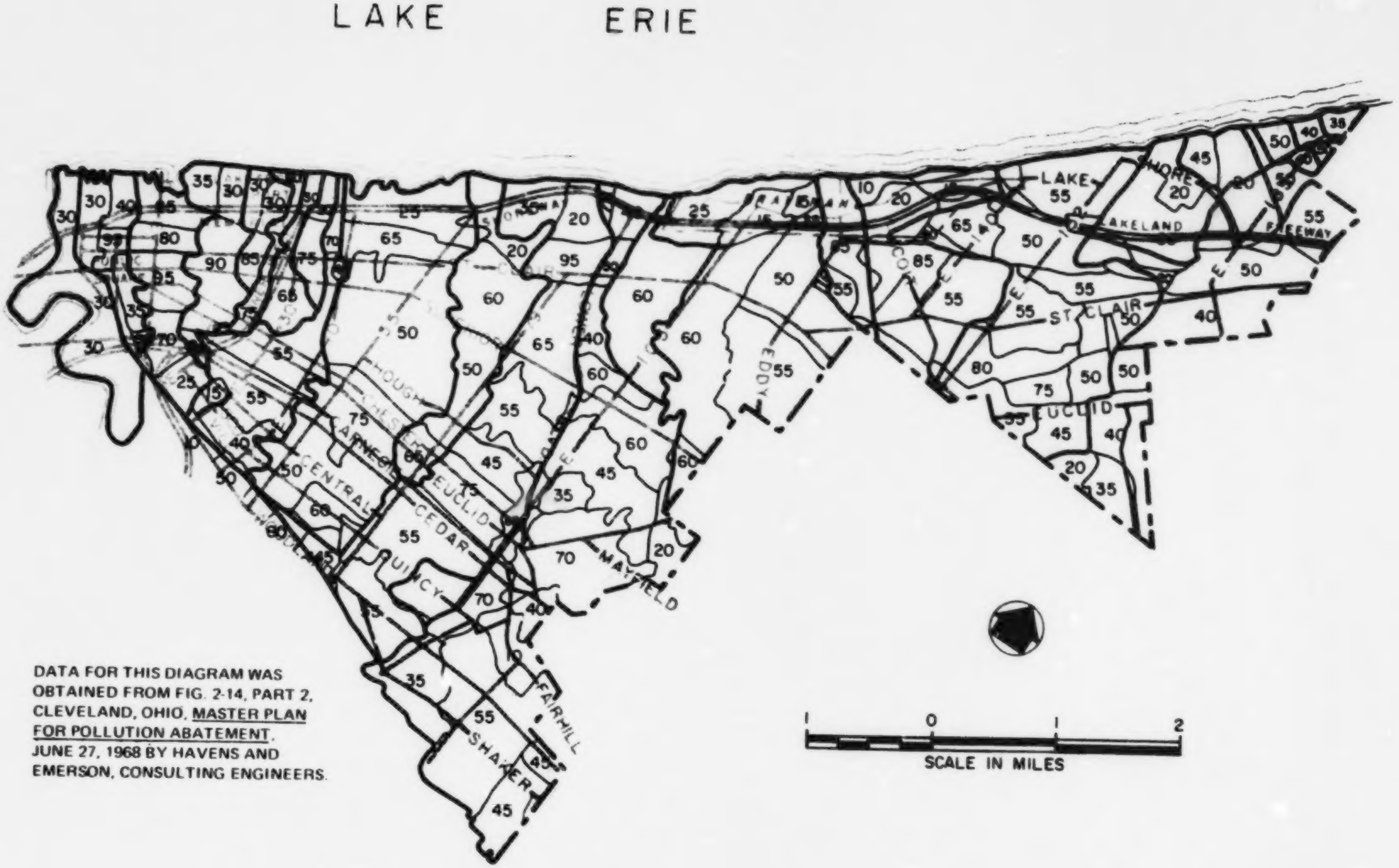
   1. impervious surfaces: 0.13
   2. pervious surface: 0.25

B. Existing Sewers

   1. Cast or ductile iron: 0.015
   2. Concrete and vitrified clay: 0.015
   3. Brick or block: 0.016
   4. Corrugated metal: 0.022

C. New Sewers (Concrete)

   1. Less than 30 in. in diameter: 0.015
   2. 30 in. to 84 in. in diameter: 0.013


LAKE ERIE
LAKE SHORE
LAKELAND
FREEWAY
ST CLAIR
EUCLID
EDDY
MAYFIELD
FAIRHILL
SHAKER
HOUGH
CHESTER
CARNEGIE
CENTRAL
CEDAR
QUINCY
WOODLAND
1
0
1
2
SCALE IN MILES

DATA FOR THIS DIAGRAM WAS OBTAINED FROM FIG. 2-14, PART 2, CLEVELAND, OHIO, MASTER PLAN FOR POLLUTION ABATEMENT, JUNE 27, 1968 BY HAVENS AND EMERSON, CONSULTING ENGINEERS.

FIG. 7 IMPERVIOUS AREAS, EASTERLY DISTRICT

        3. Greater than 84 in. in diameter: 0.011

D. Culverts

    1. Existing: 0.015
    2. New: 0.011

Velocities

    1. minimum: 3 fps (feet per second)
    2. maximum: 15 fps.

Infiltration and Dry-Weather Flow. Sample calculations show that both infiltration and dry-weather flows have negligible hydraulic significance on combined sewers during design storm conditions and may be neglected if pollution aspects are not being considered.

For example, in Area No. 8 inclusion of infiltration would increase flows at the downstream end by less than 0.2 percent.

Similarly, selecting Subcatchment No. 1 in Area No. 8 inclusion of dry-weather flow would increase design flows by under 2 percent.

Allowance for Inflow from Suburban Tributary Areas. It was necessary to develop hydrographs of inflow in certain locations where inflows from suburban communities were a significant factor in the analysis of combined sewers.

Two approaches were used to determine the quantity of dry-weather inflow from the Monticello Boulevard Sewer. The first method was based on tributary area, population and water use. The second method was based on the assumption that the peak inflow is equal to the capacity of the inflow pipe. The greater of the two values was used as a constant inflow in the determination of improvements.

In the modeling of those enclosed conduits which transport drainage from suburban areas, it was necessary to make allowances for such contributions. For each, a peak inflow and time to peak was determined from previous studies. An inflow hydrograph was developed using this peak discharge and, the shape of hydrographs generated from areas with similar basin characteristics that had been previously modeled. These inflow hydrographs were incorporated in each of the applicable drainage areas.

Model Demonstration

For the purpose of demonstrating the application of the SWMM the rainfall and flow during such rainfall from Areas 8 and 13 was gaged and such conditions were then simulated by computer for comparison.

Area No. 8 Data. The pertinent data for this area is as follows:

Size: 692.4 acres.
Impervious area proportion: 55 percent.
General ground slope: 0.3 percent
Total length of major sewer: 12,000 feet.
Size of downstream sewer: 105 inch circular brick sewer.
Capacity of downstream sewer: 660 cfs.

Area No. 13 Data. The pertinent data for this area is as follows:

Size: 376.0 acres.
Impervious area proportion: 60 percent.
General ground slope: 0.35 percent.

Total length of major sewer: 7,200 feet.
Size of downstream sewer: 87 inches.
Capacity of downstream sewer: 285 cfs.

Rainfall Data. The precipitation applicable to flows measured in areas 8 and 13 was recorded by rainfall gages located near the drainage areas. These gages recorded the cumulative rainfall every ten minutes providing data for the complete rainfall hyetographs shown on Figure 8. As shown, there is a large variability in rainfall between locations for these recorded events. The input rainfall for Area No. 8 is a weighted average of the rainfall records recorded at both gages. The rainfall recorded at the East 105th Street Gage was used directly as input to Area No. 13. The input hyetographs for each area are shown on Figures 9 and 10, respectively.

Flow Measurements. Each gage and recording meter was located in a selected manhole where it measured and recorded the depth of flow at set time intervals. The chart had a full scale reading equal to 60-inches permitting the recording of flows from zero depth (pipe invert) to a depth of 60-inches. This recorded depth was converted to a flow rate by assuming that a condition of uniform flow exists, therefore, using Manning's Equation for such conversion with a pipe friction factor (n) of 0.016.

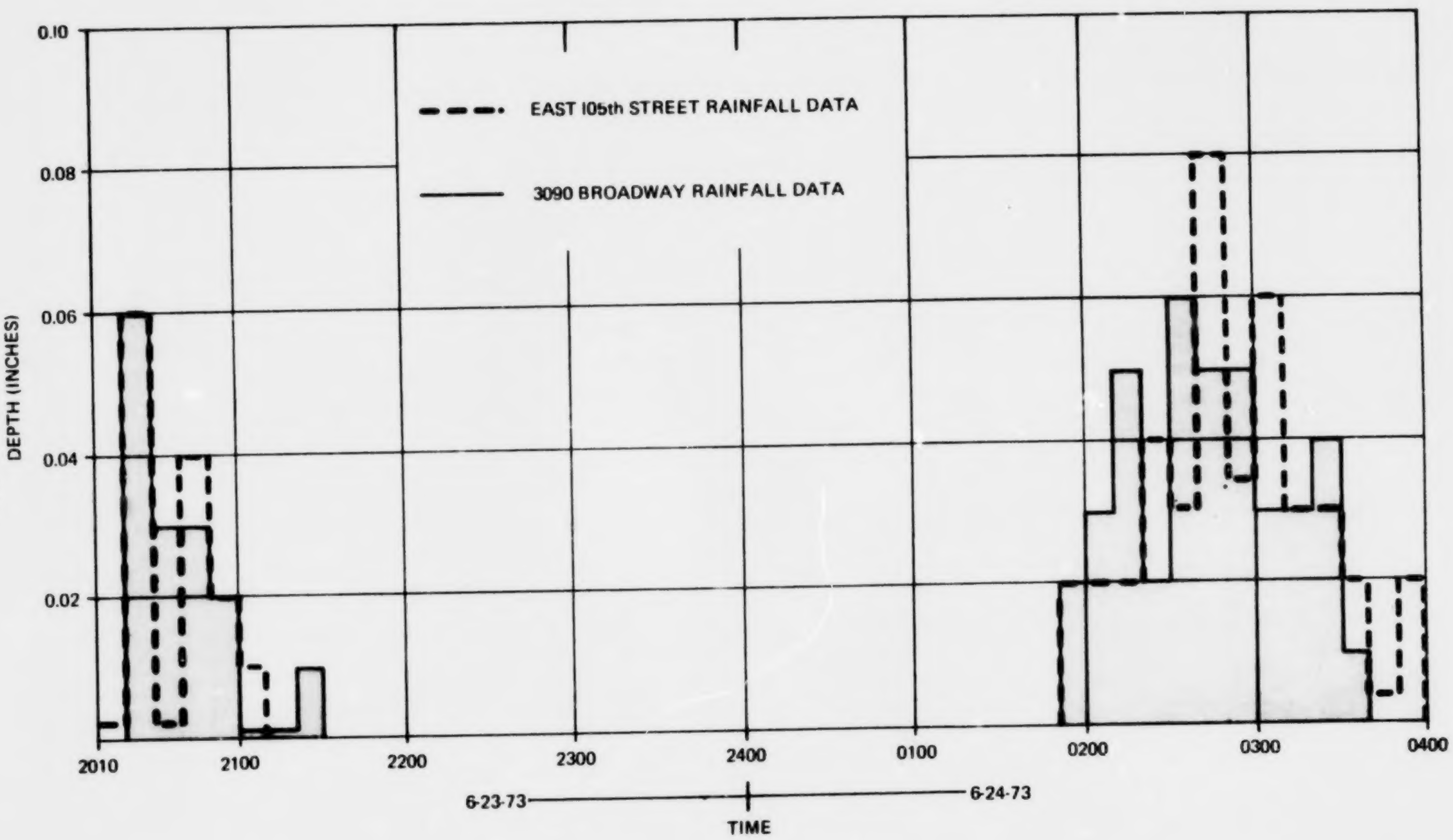


FIG. 8 RAINFALL HYETOGRAPHS, TEST AREAS, CLEVELAND, OHIO

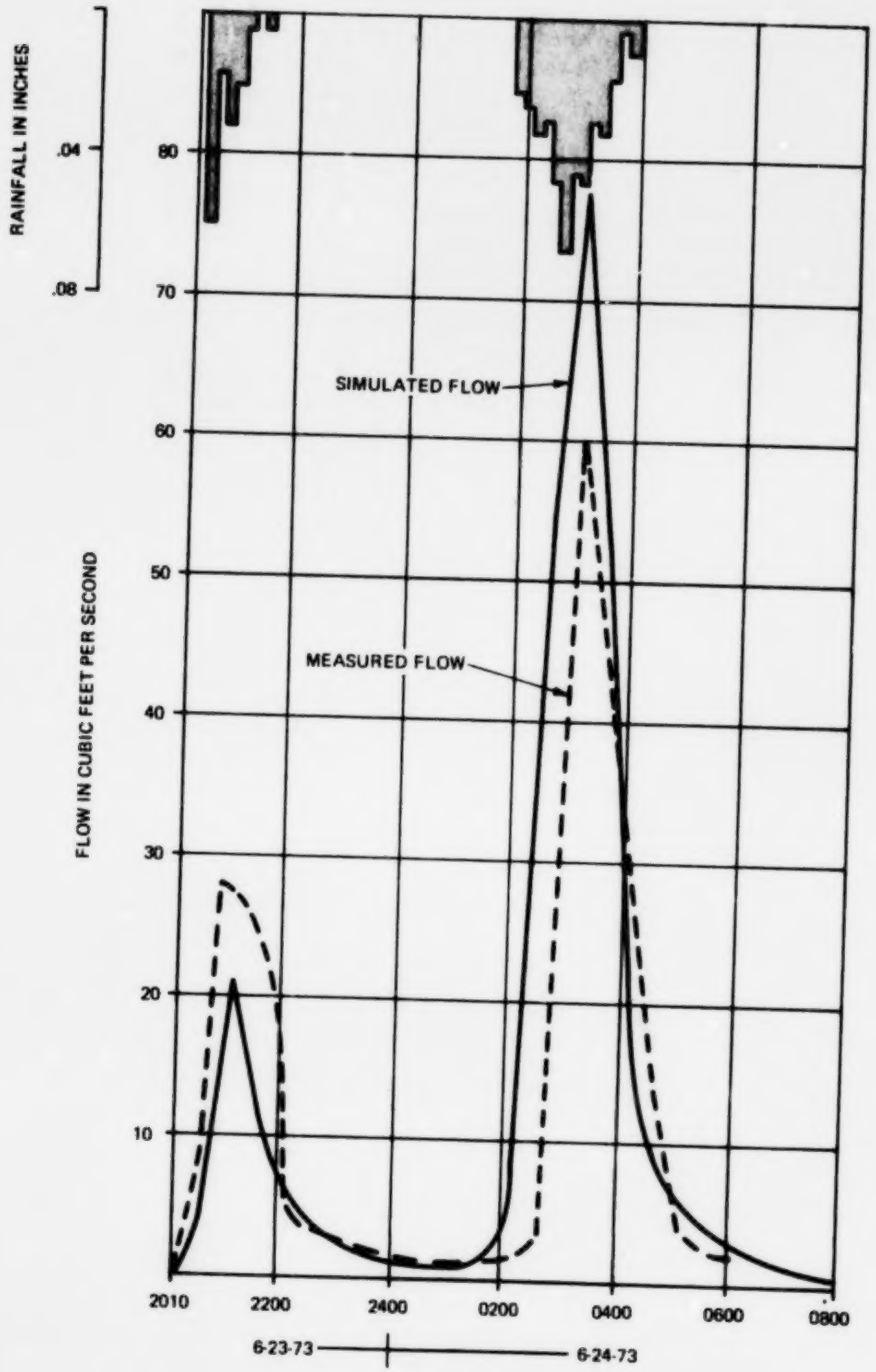


FIG. 9 MEASURED AND SIMULATED FLOWS AREA NO. 8, CLEVELAND, OHIO

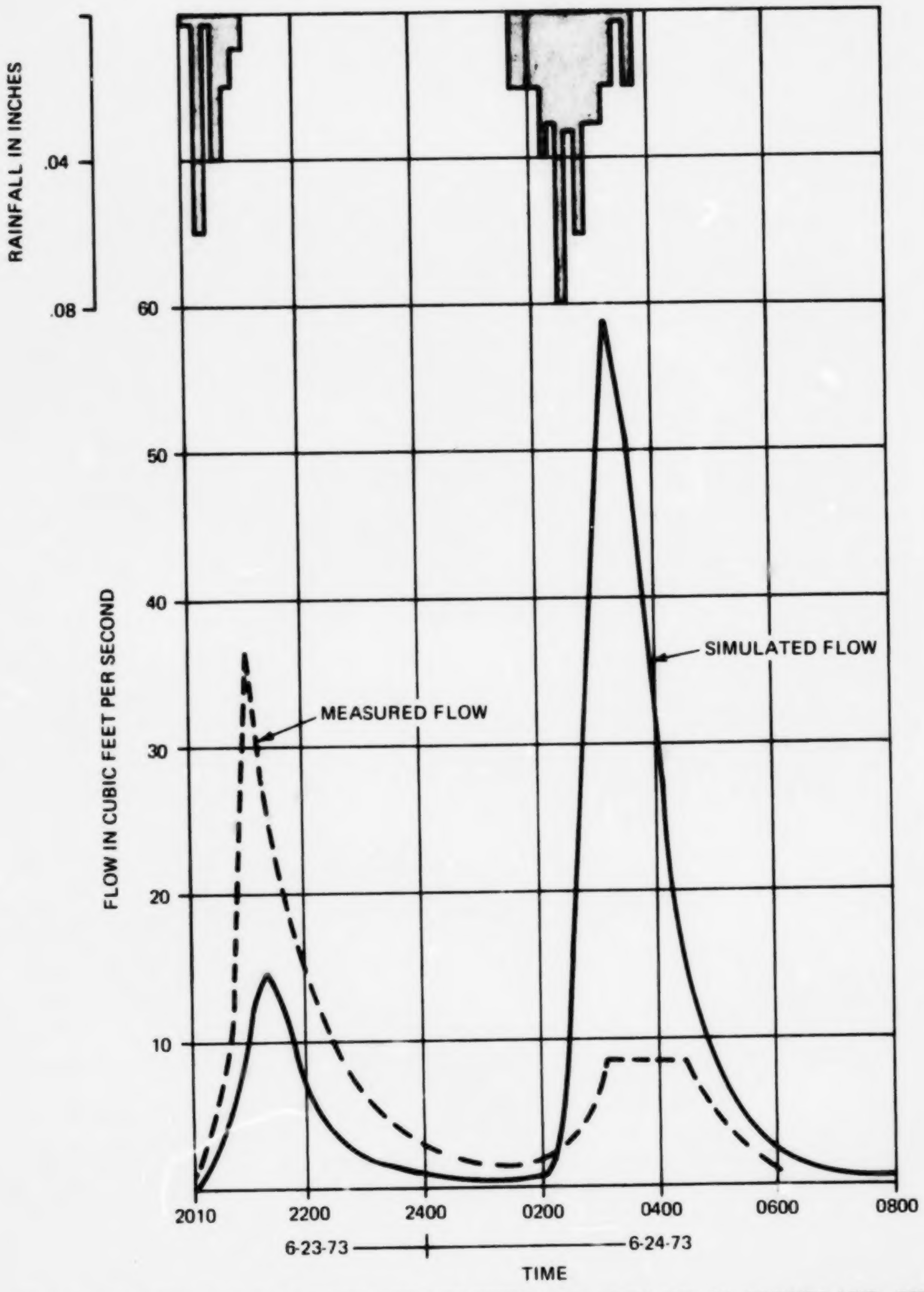


FIG. 10 MEASURED AND SIMULATED FLOWS EAST, AREA NO. 13, CLEVELAND, OHIO

Model Comparison. The SWMM was then used to simulate conditions during the recorded rainfall events of July 23rd and 24th, 1973. These results are also shown on Figures 9 and 10 for Areas No. 8 and 13, respectively.

The agreement between computed and observed values appears reasonable for Area No. 8, but is questionable for Area No. 13. Some explanations for these variations may be as follows:

Two operational problems that occur with the initial flow measurements involve: 1) the sudden recorded rise in peak flow above the normal flow; and 2) the recording of the correct time the rise in flow occurs. The first problem has been attributed to the turbulence accompanying rapid rise in the level of the flow in the pipe. An example occurs in Area No. 8 during the evening storm of June 24th. The sudden rise in the level of the flow from a Manning Chart reading of 4 to a reading of 7 was attributed to such turbulence and omitted in the analysis of the data. The second problem, proper time, can be caused by an inaccurate starting time or inaccurate clock speed. An example of this occurs at Area No. 13 during the evening storm of the 24th of June. The recorded flow peaks almost an hour before the rain gages record any precipitation. This was adjusted also on Figure 10.

Precipitation variations can cause errors between observed and computed data especially during small rainfalls, such as those measured. This problem is more pronounced in this case where the gages were not located within the drainage area. The actual rainfall distribution, duration, and intensity in Area No. 8 and Area No. 13 may not have been represented by the rain gage data from the stations at East 105th Street and 3090 Broadway as demonstrated by the variability of the two storms shown on Figure 8.

Another area requiring consideration when interpreting the observed and computed data is the nature of the individual storm events. The storms of June 23 and 24 were small storms which can be severly affected by surface storage, infiltration and other drainage basin characteristics which would be dampened in a larger storm. In the case of the storm on the evening of June 23, the maximum intensity occurred during the first time increment (10 minutes) and, because of its small value, it is sensitive to antecedent conditions and surface peculiarities.

Fine Model Development

As mentioned earlier, evaluation of the sewers in the Easterly District was divided into a two-part approach consisting of the computer modeling of all pipes 30 inches and greater, and the analysis by representative area of pipes smaller than that.

The general approach for evaluation of the smaller pipes was to generate a set of tables which could be used to determine the peak-design flows tributary to a small pipe. The parameters pertinent to this are: area size, slope and shape; percent impervious area; and design storm frequency.

Since all upstream sewers were made adequate in this analysis, the condition of a limiting sewer capacity in a specific reach did not exist. On this basis, sample areas were selected to provide the basis for selecting typical areas tributary to a small sewer system. Due to the generally orderly nature of the streets in the City, the shape was found generally rectangular with a uniform distribution of sewers.

A study of available topographic data showed that most of the Easterly District is relatively flat with a ground slope under 2.0 percent averaging at about 0.5 percent. However, there were isolated cases within the City where slopes exceed these values. These areas have slopes between 5 and 10 percent and were considered separately.

Design Table Development. Several areas were selected and modeled to generate typical peak flows for several areas under various design storms. A part of Area No. 14 was finally used as the basis for generating the design table data.

The model of this area was developed to a considerable detail in order to represent actual conditions accounting for flows on the ground, in the gutters and in pipes. A City ordinance requires that all roof drains be connected directly to the sewer system. On this basis, the runoff from roofs was modeled to go to the sewers in that street, with runoff from other areas going to the gutters in that street and then via the gutters to the next downstream sewer inlet. The fine model of this area was then developed to cover the subareas for each reach. It was estimated that 30 percent of the area consists of roofs draining this impervious area directly into the sewers (Due to computer program restrictions 95 percent imperviousness was used) with runoff from the remaining 70 percent entering gutters, and proceeding to the downstream manhole of the reach.

Allowance of 30 percent of the area as representing roofs was adhered to for all cases where the overall percent of imperviousness was over 30 percent.

Various degrees of percent imperviousness (5, 30, 65, and 95 percent) were selected and run in the fine model analysis of the sample area for the 5, 10, and 25-year frequency storms. Straight line interpolation was used to compute all intermediate points producing Tables 2 through 4.

A formula was used to separate roof areas from ground areas while retaining various overall percent imperviousness. For example, if an area was determined to have an overall percent imperviousness of 70 percent, exclusion of 30 percent of that area covered by roofs would make the remainder of the area have an imperviousness of 60 percent.

It is noted that for 0.50 acres and less, between 30 percent and 95 percent imperviousness, the tables show the runoff to be the same. Areas this small only represent the roof runoff contributing to the upstream reach and are always modeled at 95 percent imperviousness.

According to the assumption of no roof drains under 30 percent imperviousness this does not apply to less developed areas. Here it was assumed that 30 percent of its area at the actual percent imperviousness goes directly into the upstream manhole and that the other 70 percent goes into gutters and then into the downstream manhole.

Fine Model Verification. After completing the tables for the fine model analysis, a section of Area No. 13 was subdivided and tributary flows were determined. These flows were then checked against flows developed by actually modeling the same area. The results were found to be in general agreement as shown in the following table.

Representative area method Expected Q (cfs)	Computer method Expected Q (cfs)
3.69	3.57
9.32	8.68
13.00	11.51
2.55	2.54
6.44	5.63
21.46	19.31
2.18	2.19
5.72	4.86
28.68	25.87

TABLE 2. FINE MODEL ANALYSIS – 5 YEAR DESIGN STORM, FLAT AREAS
ESTIMATED FLOW (CFS) FOR VARIOUS PERCENT IMPERVIOUS AREAS

AREA (ACRES)	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95
5.60	1.68	2.99	4.30	5.61	6.92	8.23	9.32	10.41	11.50	12.59	13.68	14.77	15.85	16.87	17.89	18.90	19.92	20.94	21.95
5.70	1.72	3.05	4.37	5.69	7.02	8.34	9.45	10.56	11.67	12.78	13.89	15.00	16.11	17.16	18.20	19.25	20.29	21.34	22.38
5.80	1.76	3.10	4.43	5.77	7.11	8.45	9.58	10.71	11.85	12.98	14.11	15.24	16.37	17.45	18.52	19.59	20.66	21.74	22.81
5.90	1.79	3.15	4.50	5.85	7.20	8.55	9.71	10.86	12.02	13.17	14.33	15.48	16.63	17.73	18.83	19.93	21.03	22.13	23.23
6.00	1.83	3.20	4.56	5.93	7.30	8.66	9.84	11.01	12.19	13.37	14.54	15.72	16.89	18.02	19.15	20.28	21.41	22.53	23.66
6.10	1.87	3.25	4.63	6.01	7.39	8.77	9.97	11.16	12.36	13.56	14.76	15.96	17.15	18.31	19.47	20.62	21.78	22.93	24.09
6.20	1.90	3.30	4.69	6.09	7.48	8.87	10.09	11.31	12.53	13.75	14.97	16.19	17.41	18.60	19.78	20.96	22.15	23.33	24.51
6.30	1.94	3.35	4.76	6.17	7.57	8.98	10.22	11.47	12.71	13.95	15.19	16.43	17.67	18.89	20.10	21.31	22.52	23.73	24.94
6.40	1.98	3.40	4.82	6.24	7.67	9.09	10.35	11.62	12.88	14.14	15.41	16.67	17.93	19.17	20.41	21.65	22.89	24.13	25.37
6.50	2.02	3.45	4.89	6.32	7.76	9.20	10.48	11.77	13.05	14.34	15.62	16.91	18.20	19.46	20.73	22.00	23.26	24.53	25.80
6.60	2.05	3.50	4.95	6.41	7.86	9.31	10.62	11.92	13.23	14.53	15.84	17.14	18.45	19.72	20.99	22.26	23.53	24.80	26.08
6.70	2.08	3.55	5.02	6.49	7.96	9.43	10.76	12.08	13.41	14.73	16.05	17.38	18.70	19.98	21.25	22.53	23.80	25.08	26.36
6.80	2.11	3.60	5.09	6.58	8.07	9.55	10.90	12.24	13.58	14.93	16.27	17.61	18.95	20.23	21.51	22.79	24.07	25.35	26.63
6.90	2.14	3.65	5.16	6.66	8.17	9.67	11.04	12.40	13.76	15.12	16.48	17.85	19.21	20.49	21.78	23.06	24.35	25.63	26.91
7.00	2.17	3.70	5.22	6.75	8.27	9.79	11.18	12.56	13.94	15.32	16.70	18.08	19.46	20.75	22.04	23.33	24.62	25.91	27.19
7.10	2.21	3.75	5.29	6.83	8.37	9.91	11.31	12.71	14.11	15.51	16.91	18.31	19.71	21.01	22.30	23.59	24.89	26.18	27.47
7.20	2.24	3.80	5.36	6.92	8.48	10.03	11.45	12.87	14.29	15.71	17.13	18.55	19.97	21.27	22.56	23.86	25.16	26.46	27.75
7.30	2.27	3.85	5.42	7.00	8.58	10.15	11.59	13.03	14.47	15.91	17.35	18.78	20.22	21.52	22.83	24.13	25.43	26.73	28.03
7.40	2.30	3.90	5.49	7.09	8.68	10.27	11.73	13.19	14.65	16.10	17.56	19.02	20.47	21.78	23.09	24.39	25.70	27.01	28.31
7.50	2.33	3.95	5.56	7.17	8.78	10.39	11.87	13.35	14.82	16.30	17.78	19.25	20.73	22.04	23.35	24.66	25.97	27.28	28.59
7.60	2.37	4.00	5.63	7.26	8.89	10.51	12.01	13.51	15.00	16.50	17.99	19.49	20.98	22.30	23.61	24.93	26.24	27.56	28.87
7.70	2.40	4.05	5.69	7.34	8.99	10.63	12.15	13.66	15.18	16.69	18.21	19.72	21.23	22.55	23.87	25.19	26.51	27.83	29.15
7.80	2.43	4.10	5.76	7.43	9.09	10.75	12.29	13.82	15.35	16.89	18.42	19.95	21.49	22.81	24.14	25.46	26.79	28.11	29.43
7.90	2.46	4.15	5.83	7.51	9.19	10.87	12.43	13.96	15.53	17.08	18.64	20.19	21.74	23.07	24.40	25.73	27.06	28.39	29.71
8.00	2.50	4.20	5.90	7.60	9.30	11.00	12.57	14.14	15.71	17.28	18.85	20.42	22.00	23.33	24.66	26.00	27.33	28.66	30.00
8.10	2.55	4.27	6.00	7.73	9.46	11.20	12.77	14.35	15.93	17.51	19.09	20.67	22.25	23.60	24.96	26.32	27.68	29.04	30.40
8.20	2.59	4.35	6.11	7.87	9.63	11.39	12.98	14.57	16.15	17.74	19.32	20.91	22.50	23.88	25.26	26.64	28.03	29.41	30.79
8.30	2.64	4.43	6.22	8.01	9.80	11.59	13.19	14.78	16.37	17.97	19.56	21.15	22.75	24.15	25.56	26.97	28.38	29.79	31.19
8.40	2.69	4.51	6.33	8.15	9.97	11.79	13.39	14.99	16.59	18.19	19.79	21.39	23.00	24.43	25.86	27.29	28.73	30.16	31.59
8.50	2.74	4.59	6.44	8.29	10.14	11.99	13.60	15.21	16.82	18.42	20.03	21.64	23.25	24.70	26.16	27.62	29.08	30.54	31.99
8.60	2.79	4.67	6.55	8.43	10.31	12.19	13.81	15.42	17.04	18.65	20.27	21.88	23.50	24.98	26.46	27.94	29.43	30.91	32.39
8.70	2.84	4.75	6.66	8.57	10.48	12.39	14.02	15.64	17.26	18.88	20.50	22.12	23.75	25.25	26.76	28.27	29.78	31.29	32.79
8.80	2.89	4.83	6.77	8.71	10.65	12.59	14.22	15.85	17.48	19.11	20.74	22.37	24.00	25.53	27.06	28.59	30.13	31.66	33.19
8.90	2.94	4.91	6.88	8.85	10.82	12.79	14.43	16.07	17.70	19.34	20.97	22.61	24.25	25.80	27.36	28.92	30.48	32.04	33.59
9.00	3.00	5.00	7.00	9.00	11.00	13.00	14.64	16.28	17.92	19.57	21.21	22.85	24.50	26.08	27.66	29.25	30.83	32.41	34.00
9.10	3.05	5.05	7.06	9.07	11.08	13.10	14.77	16.44	18.11	19.78	21.45	23.12	24.80	26.40	28.00	29.59	31.19	32.79	34.40
9.20	3.09	5.11	7.13	9.15	11.17	13.19	14.89	16.59	18.29	19.99	21.69	23.39	25.10	26.71	28.33	29.94	31.56	33.18	34.79
9.30	3.14	5.17	7.20	9.23	11.26	13.29	15.02	16.75	18.48	20.21	21.94	23.67	25.40	27.03	28.66	30.29	31.93	33.56	35.19
9.40	3.19	5.23	7.27	9.31	11.35	13.39	15.15	16.91	18.67	20.42	22.18	23.94	25.70	27.34	28.99	30.64	32.29	33.94	35.59
9.50	3.24	5.29	7.34	9.39	11.44	13.49	15.28	17.07	18.85	20.64	22.42	24.21	26.00	27.66	29.33	30.99	32.66	34.33	35.99
9.60	3.29	5.35	7.41	9.47	11.53	13.59	15.41	17.22	19.04	20.85	22.67	24.48	26.29	27.98	29.66	31.34	33.03	34.71	36.39
9.70	3.34	5.41	7.48	9.55	11.62	13.69	15.54	17.38	19.22	21.07	22.91	24.75	26.59	28.29	29.99	31.69	33.39	35.09	36.79
9.80	3.39	5.47	7.55	9.63	11.71	13.79	15.67	17.54	19.41	21.28	23.15	25.02	26.89	28.61	30.33	32.04	33.76	35.48	37.19
9.90	3.44	5.53	7.62	9.71	11.80	13.89	15.79	17.69	19.59	21.49	23.39	25.29	27.19	28.93	30.66	32.39	34.13	35.86	37.59
10.00	3.50	5.60	7.69	9.79	11.89	14.00	15.92	17.85	19.78	21.71	23.64	25.57	27.50	29.25	31.00	32.75	34.50	36.25	38.00

TABLE 3. FINE MODEL ANALYSIS – 10 YEAR DESIGN STORM, FLAT AREAS
ESTIMATED FLOW (CFS) FOR VARIOUS PERCENT IMPERVIOUS AREAS

AREA (ACRES)	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95
5.60	3.44	4.62	5.80	6.99	8.17	9.35	10.61	11.86	13.12	14.37	15.63	16.88	18.13	19.23	20.32	21.41	22.51	23.60	24.69
5.70	3.51	4.70	5.90	7.09	8.29	9.48	10.76	12.04	13.32	14.60	15.88	17.16	18.44	19.56	20.67	21.78	22.90	24.01	25.13
5.80	3.58	4.79	5.99	7.20	8.40	9.61	10.91	12.22	13.53	14.83	16.14	17.44	18.75	19.88	21.02	22.15	23.29	24.43	25.56
5.90	3.65	4.87	6.09	7.30	8.52	9.73	11.07	12.40	13.73	15.06	16.39	17.72	19.05	20.21	21.37	22.52	23.68	24.84	25.99
6.00	3.73	4.95	6.18	7.41	8.63	9.86	11.22	12.58	13.93	15.29	16.65	18.00	19.36	20.54	21.72	22.89	24.07	25.25	26.43
6.10	3.80	5.04	6.28	7.51	8.75	9.99	11.37	12.75	14.14	15.52	16.90	18.29	19.67	20.87	22.07	23.26	24.46	25.66	26.86
6.20	3.87	5.12	6.37	7.62	8.87	10.11	11.52	12.93	14.34	15.75	17.16	18.57	19.97	21.19	22.41	23.63	24.85	26.07	27.29
6.30	3.95	5.21	6.47	7.72	8.98	10.24	11.68	13.11	14.54	15.98	17.41	18.85	20.28	21.52	22.76	24.00	25.25	26.49	27.73
6.40	4.02	5.29	6.56	7.83	9.10	10.37	11.83	13.29	14.75	16.21	17.67	19.13	20.59	21.85	23.11	24.37	25.64	26.90	28.16
6.50	4.10	5.38	6.65	7.93	9.21	10.50	11.98	13.47	14.95	16.44	17.92	19.41	20.90	22.18	23.46	24.75	26.03	27.31	28.60
6.60	4.15	5.46	6.76	8.06	9.36	10.66	12.17	13.68	15.19	16.70	18.21	19.72	21.24	22.52	23.81	25.09	26.38	27.67	28.95
6.70	4.21	5.54	6.86	8.18	9.51	10.83	12.36	13.90	15.43	16.97	18.50	20.04	21.58	22.86	24.15	25.44	26.73	28.02	29.31
6.80	4.27	5.62	6.96	8.31	9.65	11.00	12.55	14.11	15.67	17.23	18.79	20.35	21.92	23.21	24.50	25.79	27.09	28.38	29.67
6.90	4.33	5.70	7.07	8.43	9.80	11.16	12.75	14.33	15.92	17.50	19.09	20.67	22.26	23.55	24.85	26.14	27.44	28.74	30.03
7.00	4.39	5.78	7.17	8.55	9.94	11.33	12.94	14.55	16.16	17.77	19.38	20.99	22.60	23.89	25.19	26.49	27.79	29.09	30.39
7.10	4.45	5.86	7.27	8.68	10.09	11.49	13.13	14.76	16.40	18.03	19.67	21.30	22.94	24.24	25.54	26.84	28.15	29.45	30.75
7.20	4.51	5.94	7.37	8.80	10.23	11.66	13.32	14.98	16.64	18.30	19.96	21.62	23.28	24.58	25.89	27.19	28.50	29.81	31.11
7.30	4.57	6.03	7.48	8.93	10.38	11.83	13.51	15.20	16.88	18.56	20.25	21.93	23.62	24.92	26.23	27.54	28.85	30.16	31.47
7.40	4.63	6.11	7.58	9.05	10.52	11.99	13.70	15.41	17.12	18.83	20.54	22.25	23.96	25.27	26.58	27.89	29.21	30.52	31.83
7.50	4.69	6.19	7.68	9.17	10.67	12.16	13.89	15.63	17.36	19.09	20.83	22.56	24.30	25.61	26.93	28.24	29.56	30.88	32.19
7.60	4.75	6.27	7.78	9.30	10.81	12.33	14.09	15.84	17.60	19.36	21.12	22.88	24.64	25.95	27.27	28.59	29.91	31.23	32.55
7.70	4.81	6.35	7.89	9.42	10.96	12.49	14.28	16.06	17.84	19.63	21.41	23.19	24.98	26.30	27.62	28.94	30.27	31.59	32.91
7.80	4.87	6.43	7.99	9.55	11.10	12.66	14.47	16.28	18.08	19.89	21.70	23.51	25.32	26.64	27.97	29.29	30.62	31.95	33.27
7.90	4.93	6.51	8.09	9.67	11.25	12.83	14.66	16.49	18.33	20.16	21.99	23.82	25.66	26.98	28.31	29.64	30.97	32.30	33.63
8.00	5.00	6.60	8.20	9.79	11.39	13.00	14.85	16.71	18.57	20.42	22.28	24.14	26.00	27.33	28.66	30.00	31.33	32.66	34.00
8.10	5.05	6.67	8.30	9.93	11.56	13.20	15.07	16.94	18.81	20.68	22.55	24.42	26.30	27.65	29.01	30.37	31.73	33.09	34.45
8.20	5.09	6.75	8.42	10.07	11.73	13.39	15.28	17.17	19.05	20.94	22.82	24.71	26.60	27.98	29.36	30.74	32.13	33.51	34.90
8.30	5.14	6.83	8.52	10.21	11.90	13.59	15.49	17.39	19.29	21.19	23.09	24.99	26.90	28.30	29.71	31.12	32.53	33.94	35.34
8.40	5.19	6.91	8.63	10.35	12.07	13.79	15.71	17.62	19.54	21.45	23.37	25.28	27.20	28.63	30.06	31.49	32.93	34.36	35.79
8.50	5.24	6.99	8.74	10.49	12.24	13.99	15.92	17.85	19.78	21.71	23.64	25.57	27.50	28.95	30.41	31.87	33.33	34.79	36.24
8.60	5.29	7.07	8.85	10.63	12.41	14.19	16.14	18.08	20.02	21.97	23.91	25.85	27.79	29.28	30.76	32.24	33.73	35.21	36.69
8.70	5.34	7.15	8.96	10.77	12.58	14.39	16.35	18.31	20.27	22.22	24.18	26.14	28.09	29.60	31.11	32.62	34.13	35.64	37.14
8.80	5.39	7.23	9.07	10.91	12.75	14.59	16.57	18.54	20.51	22.48	24.45	26.42	28.39	29.93	31.46	32.99	34.53	36.06	37.59
8.90	5.44	7.31	9.18	11.05	12.92	14.79	16.78	18.77	20.75	22.74	24.72	26.71	28.69	30.25	31.81	33.37	34.93	36.49	38.04
9.00	5.50	7.40	9.30	11.20	13.10	15.00	17.00	19.00	21.00	23.00	25.00	27.00	29.00	30.58	32.16	33.75	35.33	36.91	38.50
9.10	5.57	7.48	9.40	11.31	13.23	15.15	17.17	19.19	21.21	23.23	25.25	27.27	29.30	30.90	32.51	34.12	35.73	37.34	38.95
9.20	5.64	7.57	9.50	11.43	13.36	15.30	17.34	19.38	21.42	23.47	25.51	27.55	29.60	31.23	32.86	34.50	36.13	37.76	39.40
9.30	5.71	7.65	9.60	11.55	13.50	15.45	17.51	19.57	21.64	23.70	25.77	27.83	29.90	31.55	33.21	34.87	36.53	38.19	39.84
9.40	5.77	7.74	9.70	11.67	13.63	15.60	17.68	19.77	21.85	23.94	26.02	28.11	30.20	31.88	33.56	35.24	36.93	38.61	40.29
9.50	5.84	7.82	9.80	11.78	13.76	15.75	17.85	19.96	22.07	24.17	26.28	28.39	30.50	32.20	33.91	35.62	37.33	39.04	40.74
9.60	5.91	7.91	9.91	11.90	13.90	15.89	18.02	20.15	22.28	24.41	26.54	28.67	30.79	32.53	34.26	35.99	37.73	39.46	41.19
9.70	5.98	8.00	10.01	12.02	14.03	16.04	18.19	20.34	22.49	24.64	26.79	28.94	31.09	32.85	34.61	36.37	38.13	39.89	41.64
9.80	6.05	8.08	10.11	12.14	14.17	16.19	18.37	20.54	22.71	24.88	27.05	29.22	31.39	33.18	34.96	36.74	38.53	40.31	42.09
9.90	6.12	8.17	10.21	12.26	14.30	16.34	18.54	20.73	22.92	25.12	27.31	29.50	31.69	33.50	35.31	37.12	38.93	40.74	42.54
10.00	6.20	8.26	10.31	12.37	14.43	16.50	18.71	20.92	23.14	25.35	27.57	29.78	32.00	33.83	35.66	37.49	39.33	41.16	43.00

TABLE 4. FINE MODEL ANALYSIS – 5 YEAR DESIGN STORM, STEEP AREAS
ESTIMATED FLOW (CFS) FOR VARIOUS PERCENT IMPERVIOUS AREAS

AREA (ACRES)	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95
5.70	3.81	5.02	6.23	7.44	8.65	9.87	10.92	11.96	13.01	14.06	15.11	16.16	17.22	18.22	19.22	20.23	21.23	22.24	23.25
5.80	3.90	5.11	6.32	7.54	8.75	9.96	11.09	12.22	13.36	14.49	15.62	16.75	17.88	18.82	19.76	20.70	21.64	22.58	23.52
5.90	4.00	5.21	6.42	7.63	8.84	10.06	11.27	12.48	13.70	14.91	16.13	17.34	18.56	19.43	20.30	21.17	22.04	22.91	23.79
6.00	4.07	5.28	6.49	7.70	8.91	10.12	11.34	12.56	13.78	15.01	16.23	17.45	18.67	19.59	20.52	21.44	22.37	23.29	24.22
6.10	4.14	5.35	6.56	7.77	8.98	10.19	11.42	12.65	13.87	15.10	16.32	17.55	18.78	19.76	20.74	21.72	22.70	23.68	24.66
6.20	4.21	5.42	6.63	7.84	9.05	10.26	11.49	12.73	13.96	15.19	16.42	17.66	18.89	19.92	20.96	21.99	23.02	24.06	25.09
6.30	4.29	5.49	6.70	7.91	9.12	10.33	11.57	12.81	14.05	15.28	16.52	17.76	19.00	20.09	21.18	22.26	23.35	24.44	25.53
6.40	4.36	5.57	6.77	7.98	9.19	10.40	11.64	12.89	14.13	15.38	16.62	17.87	19.11	20.25	21.40	22.54	23.68	24.82	25.96
6.50	4.43	5.64	6.85	8.05	9.26	10.47	11.72	12.97	14.22	15.47	16.72	17.97	19.22	20.42	21.62	22.81	24.01	25.20	26.40
6.60	4.51	5.71	6.92	8.12	9.33	10.54	11.79	13.05	14.31	15.56	16.82	18.08	19.34	20.59	21.84	23.09	24.34	25.59	26.84
6.70	4.54	5.77	7.00	8.23	9.46	10.69	11.97	13.24	14.52	15.80	17.07	18.35	19.63	20.91	22.18	23.46	24.74	26.01	27.29
6.80	4.58	5.83	7.08	8.34	9.59	10.84	12.14	13.44	14.74	16.03	17.33	18.63	19.92	21.23	22.53	23.83	25.14	26.44	27.74
6.90	4.61	5.89	7.17	8.45	9.72	11.00	12.32	13.63	14.95	16.27	17.58	18.90	20.22	21.55	22.88	24.21	25.54	26.67	28.20
7.00	4.65	5.95	7.25	8.55	9.85	11.15	12.49	13.83	15.16	16.50	17.84	19.17	20.51	21.87	23.22	24.58	25.94	27.29	28.65
7.10	4.69	6.01	7.34	8.66	9.99	11.31	12.67	14.02	15.38	16.73	18.09	19.45	20.80	22.19	23.57	24.95	26.34	27.72	29.10
7.20	4.72	6.07	7.42	8.77	10.12	11.46	12.84	14.22	15.59	16.97	18.35	19.72	21.10	22.51	23.92	25.33	26.74	28.15	29.56
7.30	4.76	6.13	7.50	8.88	10.25	11.62	13.02	14.41	15.81	17.20	18.60	20.00	21.39	22.83	24.26	25.70	27.14	28.57	30.01
7.40	4.60	6.19	7.59	8.98	10.38	11.78	13.19	14.61	16.02	17.44	18.85	20.27	21.69	23.15	24.61	26.08	27.54	29.00	30.47
7.50	4.84	6.25	7.66	9.07	10.49	11.90	13.34	14.78	16.22	17.65	19.09	20.53	21.97	23.45	24.93	26.41	27.89	29.38	30.86
7.60	4.88	6.31	7.74	9.17	10.60	12.03	13.49	14.95	16.41	17.87	19.33	20.79	22.25	23.75	25.25	26.75	28.25	29.75	31.25
7.70	4.92	6.37	7.81	9.26	10.71	12.15	13.64	15.12	16.60	18.09	19.57	21.05	22.54	24.05	25.57	27.09	28.61	30.12	31.64
7.80	4.96	6.42	7.89	9.35	10.81	12.28	13.78	15.29	16.80	18.30	19.81	21.32	22.82	24.36	25.89	27.43	28.96	30.50	32.03
7.90	5.00	6.48	7.96	9.44	10.92	12.40	13.93	15.46	16.99	18.52	20.05	21.58	23.11	24.66	26.21	27.76	29.32	30.87	32.42
8.00	5.04	6.54	8.04	9.53	11.03	12.53	14.08	15.63	17.18	18.74	20.29	21.84	23.39	24.96	26.53	28.10	29.67	31.24	32.81
8.10	5.09	6.60	8.11	9.63	11.14	12.66	14.23	15.80	17.38	18.95	20.53	22.10	23.68	25.26	26.85	28.44	30.03	31.62	33.21
8.20	5.15	6.67	8.18	9.70	11.21	12.73	14.33	15.94	17.55	19.15	20.76	22.36	23.97	25.58	27.20	28.81	30.43	32.04	33.66
8.30	5.22	6.73	8.25	9.77	11.29	12.80	14.44	16.08	17.71	19.35	20.99	22.62	24.26	25.90	27.54	29.18	30.83	32.47	34.11
8.40	5.28	6.80	8.32	9.84	11.36	12.88	14.54	16.21	17.88	19.55	21.22	22.88	24.55	26.22	27.89	29.56	31.23	32.89	34.56
8.50	5.35	6.87	8.39	9.91	11.43	12.95	14.65	16.35	18.05	19.75	21.45	23.15	24.84	26.54	28.23	29.93	31.62	33.32	35.01
8.60	5.42	6.94	8.46	9.98	11.50	13.02	14.75	16.48	18.22	19.95	21.68	23.41	25.14	26.86	28.58	30.30	32.02	33.74	35.47
8.70	5.48	7.01	8.53	10.05	11.57	13.10	14.86	16.62	18.38	20.14	21.91	23.67	25.43	27.18	28.93	30.67	32.42	34.17	35.92
8.80	5.55	7.07	8.60	10.12	11.65	13.17	14.96	16.76	18.55	20.34	22.14	23.93	25.72	27.50	29.27	31.05	32.82	34.60	36.37
8.90	5.61	7.14	8.67	10.19	11.72	13.25	15.07	16.89	18.72	20.54	22.37	24.19	26.01	27.82	29.62	31.42	33.22	35.02	36.82
9.00	5.68	7.21	8.74	10.26	11.79	13.32	15.17	17.03	18.89	20.74	22.60	24.45	26.31	26.13	29.96	31.79	33.62	35.45	37.28
9.10	5.75	7.28	8.81	10.33	11.86	13.39	15.28	17.17	19.05	20.94	22.83	24.71	26.60	28.45	30.31	32.16	34.02	35.87	37.73
9.20	5.81	7.34	8.87	10.41	11.94	13.47	15.38	17.30	19.22	21.14	23.06	24.97	26.89	28.77	30.65	32.54	34.42	36.30	38.18
9.30	5.88	7.41	8.94	10.48	12.01	13.54	15.49	17.44	19.39	21.34	23.29	25.23	27.18	29.09	31.00	32.91	34.82	36.72	38.63
9.40	5.95	7.48	9.01	10.55	12.08	13.62	15.60	17.58	19.56	21.54	23.52	25.50	27.48	29.41	31.34	33.28	35.21	37.15	39.09
9.50	6.01	7.56	9.12	10.68	12.23	13.79	15.79	17.78	19.78	21.78	23.77	25.77	27.76	29.72	31.67	33.62	35.57	37.53	39.48
9.60	6.07	7.65	9.23	10.81	12.39	13.97	15.98	17.99	20.00	22.02	24.03	26.04	28.05	30.02	31.99	33.96	35.93	37.90	39.87
9.70	6.13	7.73	9.33	10.94	12.54	14.14	16.17	18.20	20.23	22.26	24.28	26.31	28.34	30.33	32.31	34.30	36.29	38.28	40.27
9.80	6.19	7.82	9.44	11.07	12.70	14.32	16.37	18.41	20.45	22.50	24.54	26.58	28.63	30.63	32.64	34.64	36.65	38.65	40.66
9.90	6.25	7.90	9.55	11.20	12.85	14.50	16.56	18.62	20.68	22.74	24.80	26.85	28.91	30.94	32.96	34.98	37.01	39.03	41.05
10.00	6.31	7.98	9.66	11.33	13.00	14.67	16.75	18.83	20.90	22.98	25.05	27.13	29.20	31.24	33.28	35.32	37.36	39.40	41.45

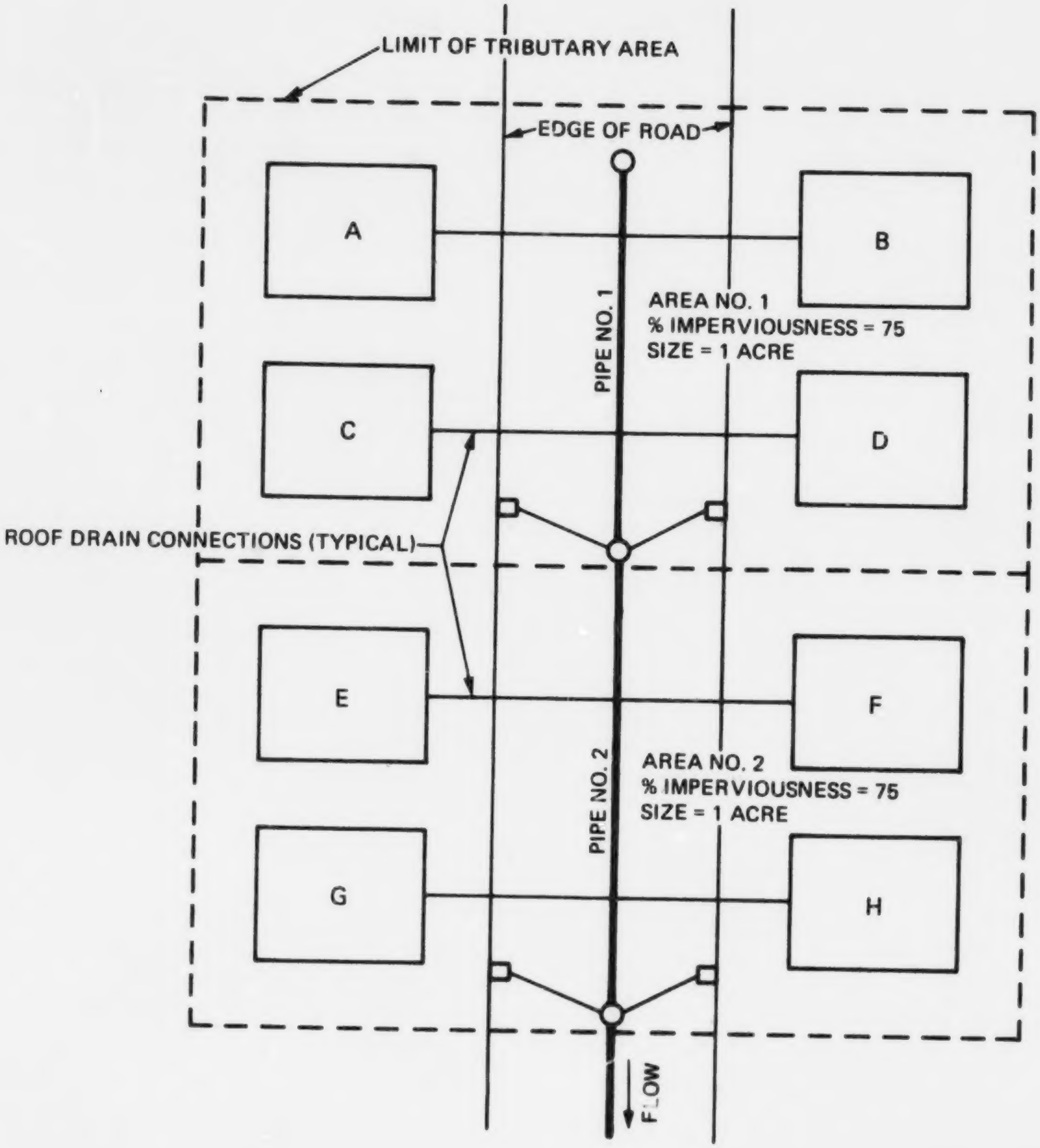

LIMIT OF TRIBUTARY AREA
EDGE OF ROAD
A
B
PIPE NO. 1
AREA NO. 1
% IMPERVIOUSNESS = 75
SIZE = 1 ACRE
C
D
ROOF DRAIN CONNECTIONS (TYPICAL)
E
F
PIPE NO. 2
AREA NO. 2
% IMPERVIOUSNESS = 75
SIZE = 1 ACRE
G
H
FLOW

FIG. 11 FINE MODEL APPLICATION

Fine Model Application. The set of tables were then used for the determination of improvements needed for the areas presently served by sewers smaller than 30 inches in diameter. In using these tables, procedures must be used that are in accordance with their development. Figure 11 is included to demonstrate the procedure which is as follows.

The area tributary to Pipe No. 1 consists of the area of the roofs of houses A, B, C, and D which discharge their roof drains directly to the sewer. The above tables show this as one part of the total area tributary to Pipe No. 1 (0.30 acres) at 95 percent imperviousness. The area tributary to Pipe No. 2 consists of the entire Area No. 1 and the roofs of houses E, F, G, and H. This is shown in the tables as 1.30 acres at 75 percent imperviousness.

CASE STUDY - 2 COMBINED SEWER OVERFLOW REGULATION IN THE CITY OF MIDDLETOWN, OHIO

Introduction

The combined sewer area in the City of Middletown consists of six drainage districts, three of which also receive sanitary inflow from separate sewers tributary to them. These drainage districts are as shown on Figure 12 and their related tributary areas are as follows:

Drainage district No.	Area tributary to combined sewers, acres	Area tributary to separate sewers upstream, acres
1	234	-
2	496	246
3	1,006	326
4	83	-
5	902	-
6	714	1,233

The districts drain in a westerly direction toward the Great Miami River. A combined sewer interceptor, located along the river, collects and transports dry-weather flows from these areas to Middletown's water pollution control plant. This plant is designed to provide secondary treatment for flows up to the capacity of the combined sewer interceptor.

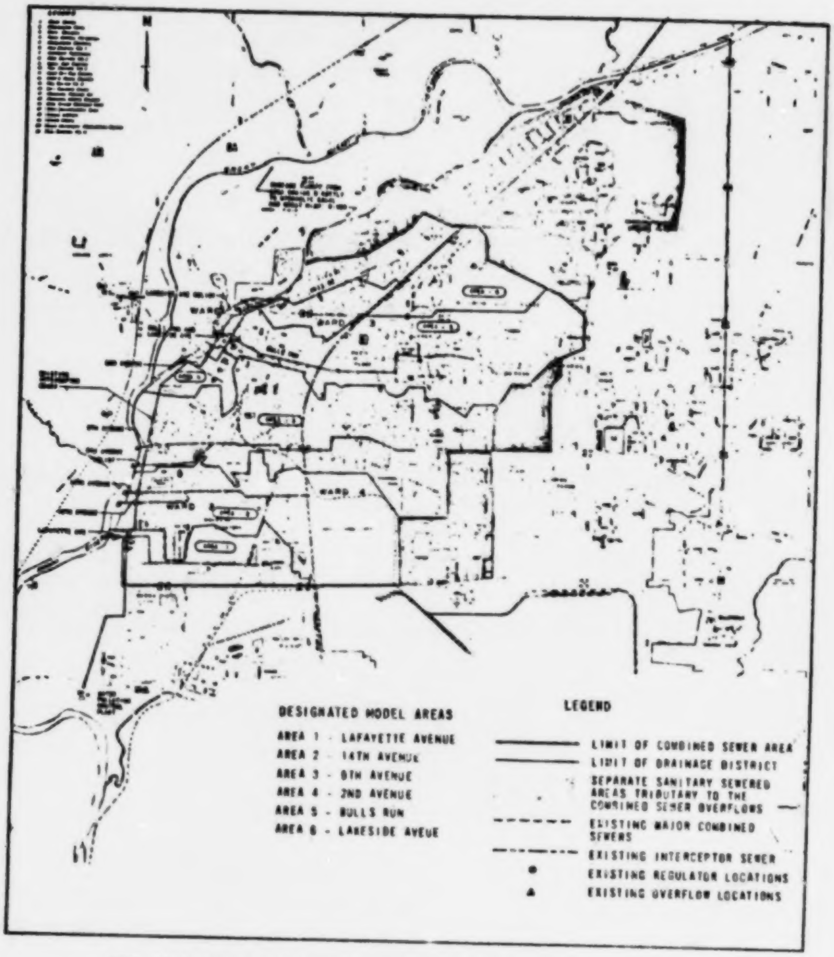

DESIGNATED MODEL AREAS
AREA 1 - LAFAYETTE AVENUE
AREA 2 - 14TH AVENUE
AREA 3 - 6TH AVENUE
AREA 4 - 2ND AVENUE
AREA 5 - BULLS RUN
AREA 6 - LAKESIDE AVEUE
LEGEND
LIMIT OF COMBINED SEWER AREA
LIMIT OF DRAINAGE DISTRICT
SEPARATE SANITARY SEWERED AREAS TRIBUTARY TO THE COMBINED SEWER OVERFLOWS
EXISTING MAJOR COMBINED SEWERS
EXISTING INTERCEPTOR SEWER
EXISTING REGULATOR LOCATIONS
EXISTING OVERFLOW LOCATIONS
WARD 4

FIG. 12 MAJOR DRAINAGE DISTRICTS, CITY OF MIDDLETOWN, OHIO

Reproduced from best available copy.

All combined sewer outfalls are connected to the interceptor by junction chambers designed to regulate flows so that the major portion of flow during storm runoff passes directly to the Great Miami River. All junction chambers are equipped with float-type regulators except one, which has a fixed orifice regulator.

In concept, the regulators should minimize the amount of flow discharging to the river by diverting the maximum possible flow to the treatment plant. Flow records at the Water Pollution Control Plant sometimes show a reduction of flow to the plant during periods of rainfall indicating that the regulators, as presently operated, perform a negative function for water pollution control.

Studies conducted during the period 1968-71, established that dissolved oxygen is the most critical problem on the Great Miami River when evaluated in terms of established water quality standards.

The purpose of this investigation was to conduct a comprehensive engineering evaluation with respect to abatement of pollution from combined sewer overflows to the Great Miami River within the City of Middletown.

Dry-Weather Flows

Dry-weather flow is comprised of domestic sewage from residences, commercial, and institutional establishments and industries; process flows from the connected industrial establishments; and infiltration that enters the sewer system.

Industrial Process Flows. The major industrial process wastewaters in the combined sewer area that are expected to contribute to the public system were obtained from the industries in terms of flow, BOD and SS and their variability.

Infiltration. Infiltration was assumed to be free of pollutants.

Sewage Flows. Values selected as an allowance for sewage flow are shown in Table 5 and were obtained from analysis of water records and treatment plant data. Plant records were analyzed in order to determine an allowance for BOD and SS contributions in dry-weather flow. These were taken excluding data during storm conditions. In order to arrive at a representative value of coliforms, samples were taken from the combined sewer interceptor during times of no rainfall and were analyzed.

TABLE 5. AVERAGE SEWAGE FLOW ALLOWANCES IN MIDDLETOWN, OHIO

Item	Per capita allowance
Flow	85 gcd
BOD_5	244 mg/L
SS	150 mg/L
Coliforms	6.1×10^6 most probable number

Based on the previous criteria, the average dry-weather flow discharged from each area of the combined sewer system and its tributary upstream areas is as shown in Table 6.

TABLE 6. AVERAGE DRY-WEATHER FLOWS DISCHARGING INTO COMBINED SEWERS IN MIDDLETOWN, OHIO

Area No.	Industrial process flow,(1) mgd	Infiltration, mgd	Sewage flow, mgd
1	-	0.070	0.158
2	-	0.220	0.463
3	1.0	0.400	1.034
4	-	0.025	0.044
5	-	0.223	0.659
6	0.95	0.579	0.902
Total	1.95	1.517	3.260

1. A major portion of the industrial wastes are in the process of being diverted directly to the combined sewer interceptor.

Adjustment Factors. Adjustment factors are based on 1972 records as shown in Table 7 for daily values of flow, BOD and SS. For coliform, we have assumed no variation from day to day.

TABLE 7. DAILY ADJUSTMENT FACTORS

Day of week	Flow	BOD	SS
Sunday	0.91	1.01	0.83
Monday	1.05	1.09	1.11
Tuesday	1.04	0.99	1.01
Wednesday	1.05	0.99	1.09
Thursday	1.00	0.92	0.96
Friday	1.01	1.00	1.00
Saturday	0.94	1.00	1.00

By analysis of the Manning charts which are recorded on a daily basis at the treatment plant, for dry-weather flow conditions only, it was possible to develop hourly adjustment factors for flow at the plant. These figures are presented in Table 8.

TABLE 8. HOURLY ADJUSTMENT FACTORS FOR FLOW

Hour of day	Adjustment factor	Flow represented, mgd
1	1.01	9.71
2	0.97	9.32
3	0.90	8.69
4	0.85	8.19
5	0.82	7.89
6	0.80	7.69
7	0.80	7.71
8	0.81	7.79
9	0.83	7.96
10	0.95	9.12
11	1.03	9.89
12 (noon)	1.11	10.64
13	1.14	10.92
14	1.14	11.01
15	1.14	10.92
16	1.14	11.00
17	1.15	11.10
18	1.15	11.10
19	1.10	10.57

TABLE 8 (Continued). HOURLY ADJUSTMENT FACTORS FOR FLOW

Hour of day	Adjustment factor	Flow represented, mgd
20	1.08	10.41
21	1.08	10.36
22	1.04	10.04
23	1.01	9.75
24 (midnight)	1.01	9.70

Since BOD, SS, and coliforms are not normally sampled and tested on an hourly basis in Middletown, data from nearby Cincinnati were used.

Wet-Weather Flows

In selecting the storm events to be used in this study, hourly rainfall records were obtained from the United States Department of Commerce, National Weather Records Center, for Oxford and Dayton and daily averages were obtained for Middletown.

These storm records were analyzed and compared in terms of duration, frequency of occurrence, intensity, and total and average monthly rainfall.

Design Storm Definition. A period of six hours with no measured rain was selected to separate each event. On this basis, storms start with the first measurable rainfall after six consecutive dry hours and end at the next six consecutive hours of no measured rain. The period of separation was judgmental and of no special hydrologic significance. It was selected as a practical time needed for system dewatering.

A separation time other than six hours would change statistics associated with the data only in terms of the number of storms and the total rainfall per storm. It would not change the intensity of a storm or the total rainfall per year.

Rainfall Data Analysis. Total and summer rainfall events were ranked by maximum hourly rainfall, total rainfall, and duration.

Summer storms (those storms occurring between June 1 and September 30) were treated separately to determine how storm characteristics change during the critical water quality season.

The maximum expected hourly intensity occurring once per year is about 1.1 inches per hour; the maximum total expected rainfall event per year is about 2.5 inches; and on the average, 46 storms events per year are expected to last at least six hours.

Rainfall data compiled in Technical Paper No. 40 for the Middletown area compares well with the data developed. For this reason, intensity ratios derived from Technical Paper No. 40 for durations of less than 60 minutes were used for design hyetograph development. The procedure used is similar to that described in Case Study No. 1. The one-year design hyetograph is shown on Figure 13.

Surface Runoff Quality. Surface runoff, once considered as clean water, contains a relatively large composition of contaminants which cannot be ignored in water quality control management. According to recent studies,* the magnitude of the three pollution components considered in this report are characterized as follows:

> BOD_5 content of runoff equals about the strength of domestic sewage after secondary treatment from the same land use.
>
> SS content of runoff is generally about three times that of untreated sewage, but consisting mostly of inorganic materials.
>
> Coliform content of runoff is about two to four orders of magnitude smaller than untreated sewage. However, it is two to five orders of magnitude higher than is considered safe for water contact recreation.

In the modeling of the combined sewer system, allowances were made for pollution from urban runoff in addition to that contributed by dry-weather flows. Allowances were based on general data recommended in the SWMM using Middletown's street-cleaning practices and design standards along with land-use data as criteria defining the amounts to be expected.

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*Urban Stormwater Management and Technology: An Assessment, December 1973, Office of Research and Development, U.S. Environmental Protection Agency by Metcalf & Eddy Engineers. (Draft)

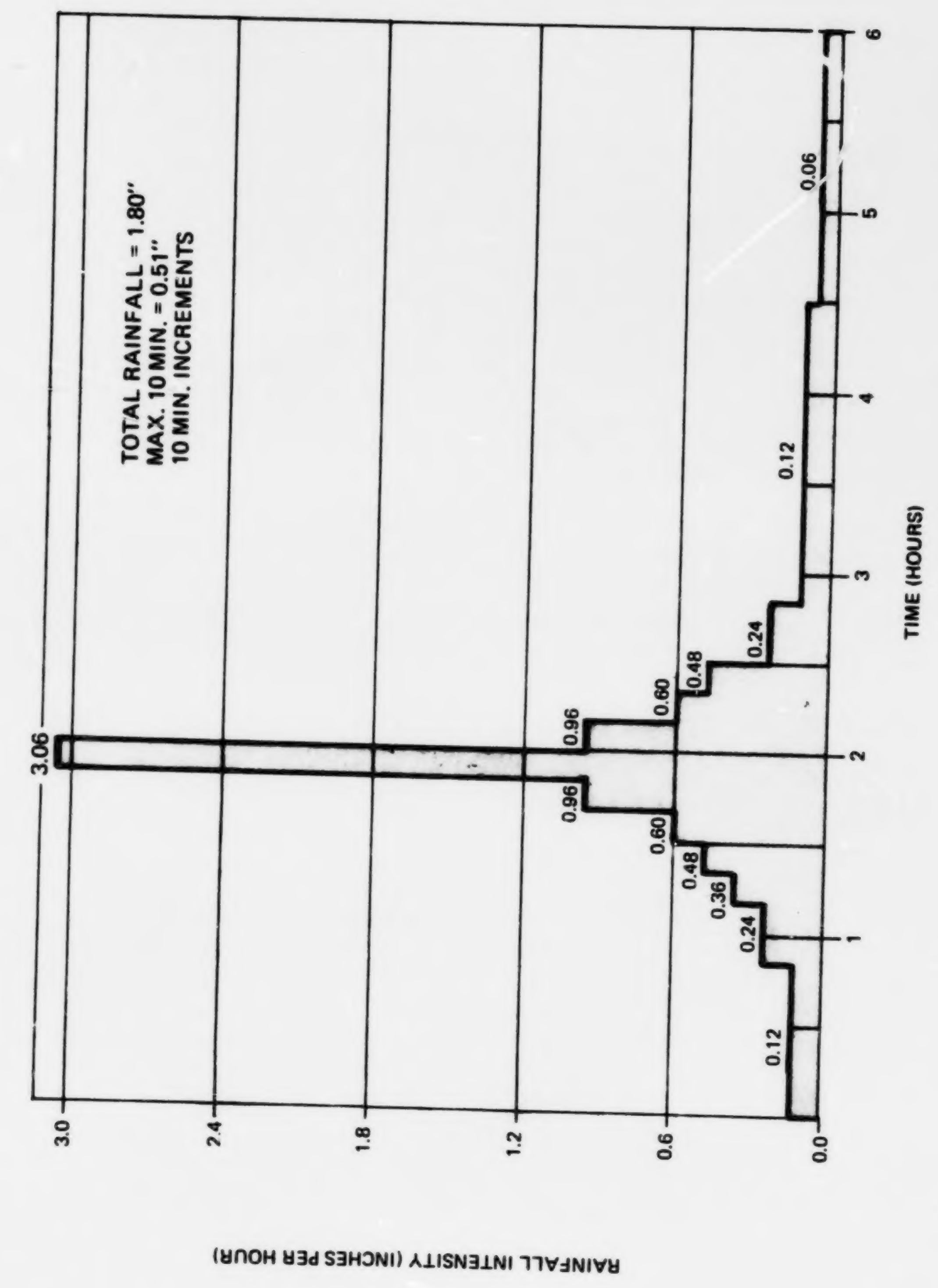

FIG. 13 1-YEAR – 6-HOUR DESIGN HYETOGRAPH FOR MIDDLETOWN, OHIO

Evaluation of Existing System

The total capacity of the combined sewer system on the basis of the capacity of the outfalls is 1,810 cfs. For each drainage area, this is shown in Table 9.

TABLE 9. ONE-YEAR DESIGN STORM OUTFLOWS

Area No.	Full flow capacity of outfalls, cfs	Peak flow(1), cfs	BOD_5(2), lb	SS(2), lb	Coliforms(1), most probable number/100 ml
1	65	63	260	3,590	6.62×10^6
2(15)	85	45	150	1,190	5.87×10^6
2(14)	198	171	670	7,200	5.87×10^6
3(9)	147	182	1,710	12,720	7.23×10^6
3(relief)	480	385	540	8,130	8.31×10^5
4	71	69	110	1,180	2.68×10^6
5	456	533	1,010	10,430	5.66×10^6
6	308	364	2,320	9,850	6.95×10^6

1. Maximum rate.
2. Total for entire storm.

In order to determine the amounts of flow and pollutants being discharged from each area under storm conditions, computer simulation was used. The results for the one-year design storm are shown in Table 9.

Combined Sewer Interceptor. In order that the operation of overflow regulators could be evaluated appropriately, a detailed hydraulic analysis of the combined sewer interceptor was carried out under steady state conditions.

Regulators. The float-operated, self-powered regulators are housed in twin below-ground vaults located along one side of a relief sewer. The downstream chamber houses the float and is connected to the relief sewer by a telltale conduit. The water level in the relief sewer controls the float movement which, in turn, controls the regulator gate located in the upstream chamber. The float is connected to the regulator sluice gate by a sprocket and chain assembly. A low-profile invert dam located in the relief sewer diverts dry-weather flow through the regulator gate into the interceptor. As depth of flow in the relief sewer increases, the float is raised and the regulator gate is closed causing flow to be routed to the Great Miami River.

Water Pollution Control Plant. Middletown's Water Pollution Control Plant has been designed to provide secondary treatment with an average of 26.6 cfs and a maximum of 56.3 cfs allocated to the area served by the combined sewer interceptor.

Analysis of Existing System Operation. On the average, there are about 90 overflows per year from the existing system. This is based on the premise that once a storm runoff occurs in excess of the regulator capacity, regulators close bypassing all flows to the Great Miami River.

In studying the effects of 1/2-year and 1-year storms it was found that such storms with a severity sufficient to flush the street surfaces and to flush out a combined sewer system will cause nearly equal pollution loads.

The City has been farsighted in its use of storage basins in the upper reaches of the drainage area. These facilities retain runoff and reduce peak flows in the combined sewer system. However, little use has been made of the storage capabilities in the large pipes of the sewerage system itself to retain the pollution loads from the small, but frequent, rainfall events for discharge to the water pollution control facilities.

The regulator operation as designed does not permit taking maximum advantage of the present treatment plant and interceptor capacities.

Although it was not the purpose of this investigation to evaluate drainage problems, the SWMM identified areas of flooding problems supporting the City's flooding experiences.

Development of Alternatives

At the outset of the project, three objectives were established in developing alternative solutions to the abatement of pollution as a result of combined sewer overflows to the Great Miami River.

The first objective was to initially take maximum advantage of the presently existing facilities, such as the storage available in the combined sewers, the transport capacity of the interceptor, and the treatment capacity at the water pollution control plant.

The second objective was to search out procedures for additional abatement of pollution from overflows as may be required by the Ohio Environmental Protection Agency.

The third objective was to look for remedial solutions that do not require sophisticated equipment, nor specially qualified personnel to operate, nor were based on limited pilot-plant studies.

As a result of these objectives, a two-phase solution for the abatement of overflow pollution was established. Phase One consists of modifications to the existing system to the extent of taking maximum advantage of existing facilities with Phase Two consisting of providing further pollution abatement.

Remedial Opportunities. The major categories of available improvements are sewer separation; overflow containment; flow through overflow treatment without detention; and overflow treatment with detention. Each of these methods is discussed briefly as follows:

Sewer Separation. Sewer separation would cost on the order of $38 million for Middletown. Separation does not eliminate all the pollution because discharge resulting from urban runoff includes pollutants stored on streets and parking lots and in catchbasins and pipes. As mentioned previously, the quality of surface runoff discharged is generally equal to that of dry-weather flow after secondary treatment in terms of BOD and equal to three times that of dry-weather flow in terms of SS. In addition to these, a considerable amount of floating visual pollution is discharged. The inconvenience caused by the tearing up of every street to provide for sewer separation cannot be expressed in terms of cost.

Overflow Containment. Containment of overflows for the purposes of ultimate treatment and discharge is an effective method of pollution abatement. However, total containment without overflows is very costly and requires large land areas. The requirements for total containment of the one-year and half-year design storms is as follows:

	One-year storm	Half-year storm
Total off-line volume required, cf	4,973,600	2,888,500
Area, acres*	7.6	4.4

---

*The area required is based on storing 333,700 cf in the combined sewer system, 2,167,500 cf in the outfall collector conduit, diverting 1,003,000 cf to the treatment plant, and storing the remaining volume in a 15-foot deep reservoir.

Flow-Through Treatment. Treatment of overflows without any storage are included in this category using devices for screening out of pollutants or their separation from runoff by concentration techniques. Although much research is being conducted in this area, little experience exists at this time on other than test-scale projects. Secondly, without the provision for sufficient storage to provide for disinfection contact time, the elimination of bacteria cannot be achieved.

Overflow Detention and Treatment. The combination of storage and treatment was considered the most effective means for abatement of pollution for Middletown's combined sewer overflows. This consists of capturing completely the small frequent overflows for transmission to the water pollution control plant once wet-weather flows subside; and of providing for the detention, screening and disinfection of overflows during the large and less frequent storms; thereby achieving the elimination of bacteria, the removal of visual pollution and the reduction of pollutants discharged to the river.

In addition to being the most desirable in terms of current technology, facilities associated with this process are relatively easy to operate avoiding problems associated with highly sophisticated apparatus. In addition, the systems developed in this process are expandable, should requirements for additional pollution abatement become necessary. For these reasons, alternatives were based on this concept of treatment.

Various alternatives to regulator improvements, in-system storage and external storage and treatment were investigated leading to the recommended solution discussed in the next section.

Recommended Solution

The abatement of pollution from combined sewer overflows was recommended in two phases. In general, Phase One consists of making maximum use of already existing facilities for pollution abatement by diverting maximum design flows to the water pollution control plant, by retaining flows in the large combined sewers until their capacity is reached, and then by permitting excess flows to overflow into the river.

Phase Two, on the other hand, consists of further reduction of pollution by construction of facilities that would carry out disinfection, and further reduce BOD and SS discharge by capture of more flows and by treatment of all overflows.

Phase One Facilities. Phase One recommendations consist of modifying the existing regulators, adding a regulator and installing gates for in-system storage in outfall sewers serving three areas. The methods in which these solutions are recommended to be accomplished are as follows:

Regulator Modifications. The regulator system should divert the maximum pollution loads to the combined sewer interceptor in accordance with maximum design capacity of the water pollution control plant. The present regulator controls do not best accomplish this objective since they close during wet weather, thus diverting all flows to the Great Miami River.

By changing the regulator controls, a constant maximum flow can be diverted to the interceptor.

With modifications, the regulator functions as a governor and the float is actuated by the water level in the gate chamber. In this way, the diverted flow can be held within ±10 percent of the design flow.

In-System Storage. The intention of in-system storage is to maximize use of the existing sewer system to store combined sewage to the extent permitted by hydraulic conditions and to transmit this flow to the treatment facilities once space there becomes available after a storm. In-system storage serves to capture all or part of small storms and diverts them directly to the interceptor. The flow resulting from small storms, if not captured, tends to flush the combined system and transport large pollution loads to the receiving waters. Five locations appear to be available with storage totaling about 334,000 cubic feet. Of these, all except one sewer are recommended for Phase One providing a storage of 294,000 cubic feet.

In order to create in-system storage, gates are recommended to be placed in the combined system just downstream of the regulators. The gates are controlled by sensing equipment which record the water surface elevation upstream of the gate. The gate is held in the upright position (for maximum storage) during dry weather and would be lowered if it is necessary to permit large storm flows to pass in emergencies.

Phase Two Facilities. On the basis of the analysis of present discharge locations, available land, sewer system configuration and water quality, use of two combined sewage storage detention and treatment facilities are recommended, as shown on Figures 14 and 15. These have been sized to provide a minimum of 15-minutes detention time for the peak flows during a one-year design storm, thereby providing at a minimum for disinfection of overflows and for limited treatment in terms of BOD and SS reduction.

Major components associated with these facilities are shown in Table 10.

TABLE 10. MAJOR FACILITIES

Item	Areas 4, 5, and 6, Tank 1	Areas 1, 2, and 3 Tank 2
Tank		
Size	160 ft by 340 ft	100 ft by 230 ft
Volume @ 8-ft depth	473,640 cf	198,000 cf
Peak flow, one-year storm	900 cfs	750 cfs
Outfall collector conduit		
Size	10 ft-10 in. by 16 ft-3 in.	10 ft-10 in. by 16 ft-3 in.
Length	2,500 lf	3,600 lf
Volume	337,500 cf	486,000 cf

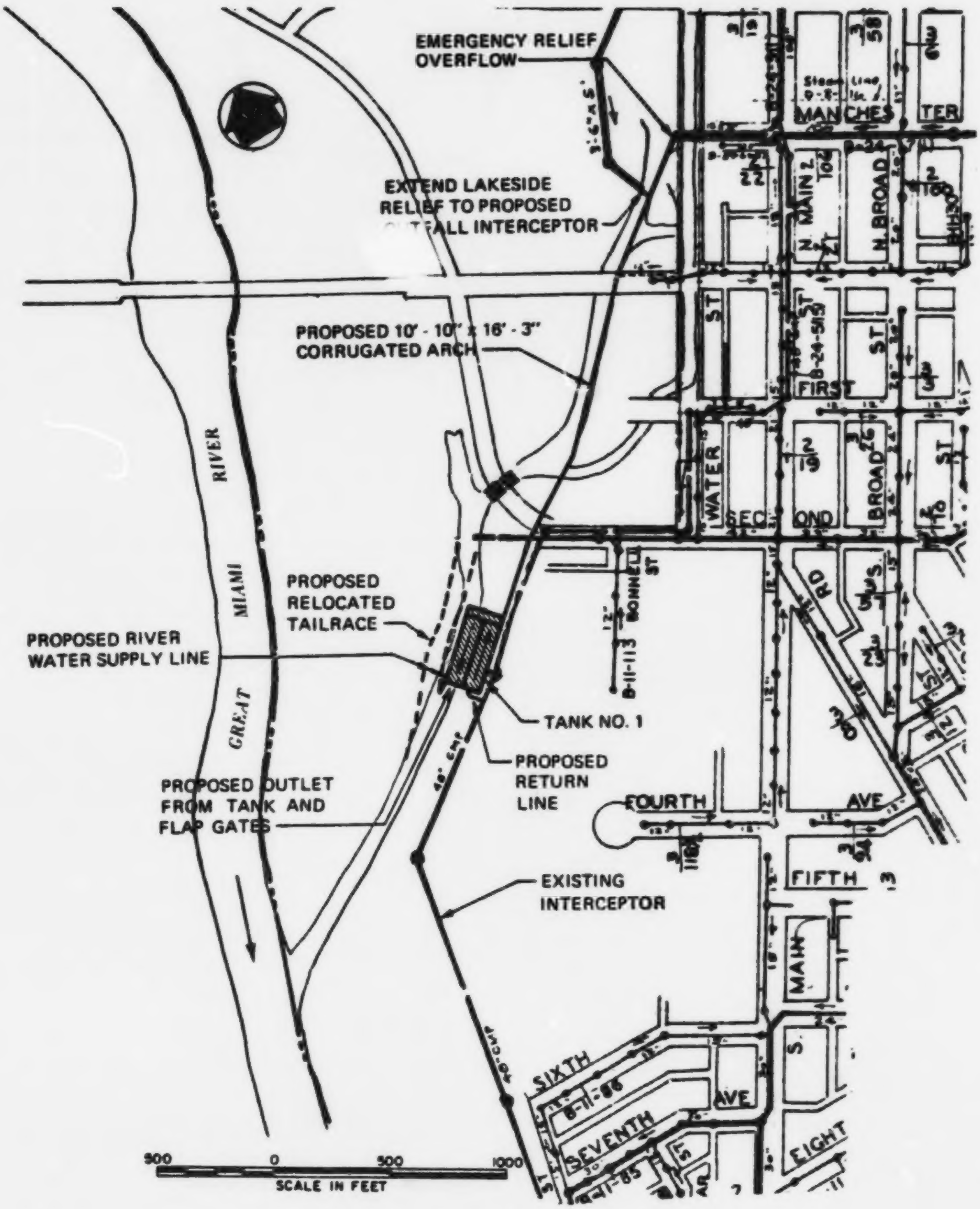
EMERGENCY RELIEF OVERFLOW
EXTEND LAKESIDE RELIEF TO PROPOSED OUTFALL INTERCEPTOR
PROPOSED 10' - 10" x 16' - 3" CORRUGATED ARCH
PROPOSED RELOCATED TAILRACE
PROPOSED RIVER WATER SUPPLY LINE
TANK NO. 1
PROPOSED RETURN LINE
PROPOSED OUTLET FROM TANK AND FLAP GATES
EXISTING INTERCEPTOR
GREAT MIAMI RIVER
MANCHESTER
N. MAIN ST
N. BROAD ST
FIRST ST
WATER ST
SECOND
BROAD
BONNELL ST
FOURTH AVE
FIFTH
MAIN
SIXTH AVE
SEVENTH
EIGHT
500 0 500 1000
SCALE IN FEET

FIG. 14 LOCATION OF PROPOSED FACILITIES TANK ONE - AREAS 4, 5, & 6, MIDDLETOWN, OHIO

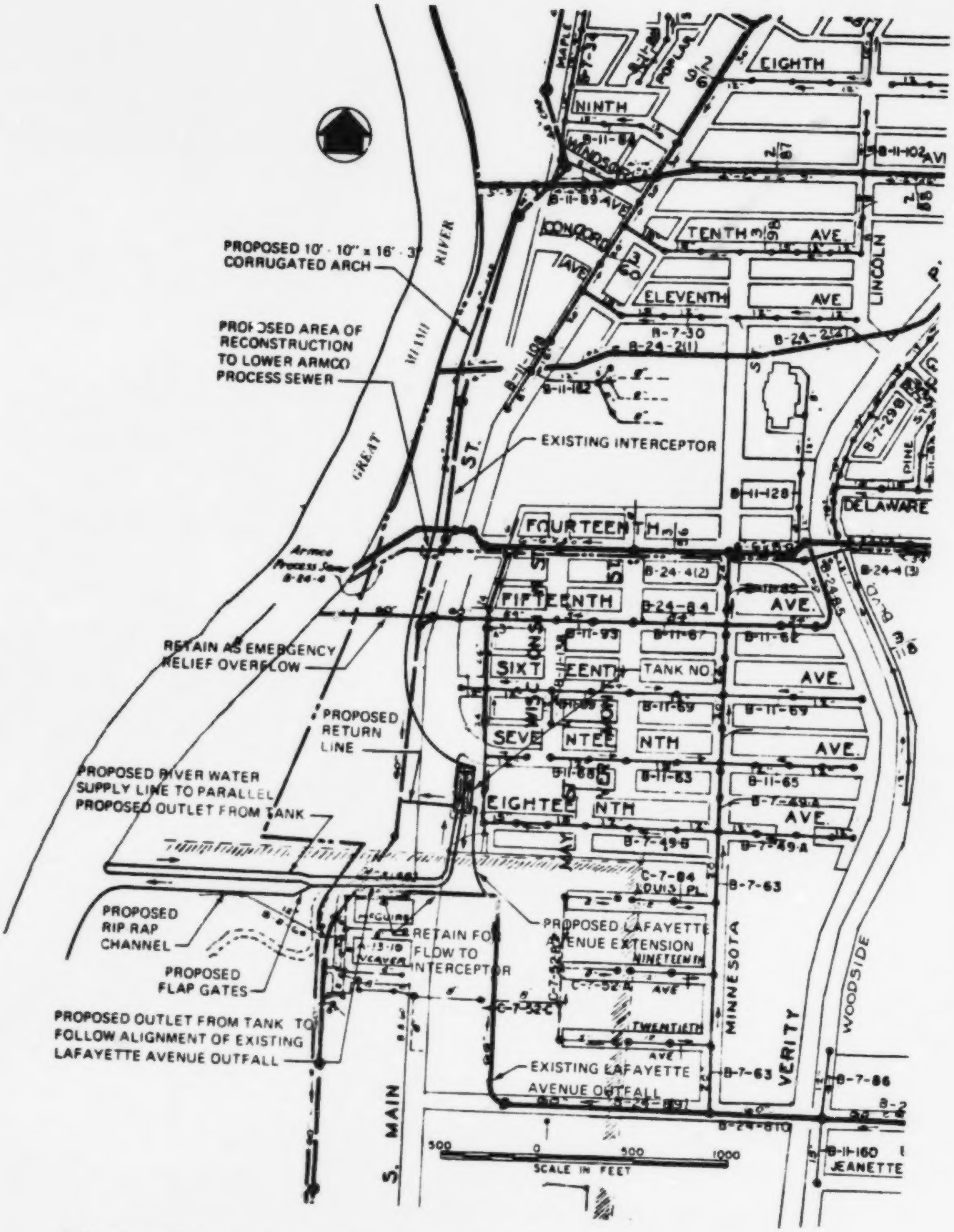
PROPOSED 10' - 10" x 16' - 3" CORRUGATED ARCH
PROPOSED AREA OF RECONSTRUCTION TO LOWER ARMCO PROCESS SEWER
EXISTING INTERCEPTOR
Armco Process Sewer B-24-6
RETAIN AS EMERGENCY RELIEF OVERFLOW
PROPOSED RETURN LINE
PROPOSED RIVER WATER SUPPLY LINE TO PARALLEL PROPOSED OUTLET FROM TANK
PROPOSED RIP RAP CHANNEL
PROPOSED FLAP GATES
PROPOSED OUTLET FROM TANK TO FOLLOW ALIGNMENT OF EXISTING LAFAYETTE AVENUE OUTFALL
RETAIN FOR FLOW TO INTERCEPTOR
PROPOSED LAFAYETTE AVENUE EXTENSION
EXISTING LAFAYETTE AVENUE OUTFALL
TANK NO. 2
GREAT MIAMI RIVER
MAIN ST.
S. MAIN
EIGHTH
NINTH
WINDSOR AVE
TENTH AVE
CONCORD AVE
ELEVENTH AVE
LINCOLN
FOURTEENTH
FIFTEENTH AVE.
SIXTEENTH AVE.
SEVENTEENTH AVE.
EIGHTEENTH AVE.
NINETEENTH AVE
TWENTIETH AVE
LOUIS PL
DELAWARE
PINE
WISCONSIN ST.
MAY ST.
MINNESOTA
VERITY
WOODSIDE BLVD.
JEANETTE
MAPLE
POPLAR
MCGUIRE
WEAVER
B-11-84
B-11-89
B-11-102
B-11-108
B-11-112
B-11-128
B-7-30
B-24-2(1)
B-24-2(2)
B-7-29 B
B-24-4(2)
B-24-4(3)
B-24-8 4
B-24-8 5
B-11-93
B-11-67
B-11-62
B-11-69
B-11-68
B-11-63
B-11-65
B-7-49 A
B-7-49 B
C-7-84
B-7-63
B-13-10
C-7-32 A
C-7-32 C
B-7-86
B-24-8 10
B-11-160
500
0
500
1000
SCALE IN FEET

FIG. 15 LOCATION OF PROPOSED FACILITIES TANK TWO - AREAS 1, 2, &3, MIDDLETOWN, OHIO

Reduction of Pollution Discharges. Since combined sewer overflows are highly variable and stormwater dependent, analysis of their effects on river pollution cannot follow the traditional approach of steady state analysis of a continuously discharged uniform waste. Two types of analysis are conducted in this report. First, an evaluation is made on the reduction in annual pollution volumes as a result of implementing Phases One and Two. Second, an evaluation is made of the effects of each phase on water quality of the Great Miami River.

Frequency of Overflows. The estimated number of overflows occurring during an average year under existing conditions and after Phases One and Two are shown in Table 11. The effect of the proposed facilities is to reduce the number of overflows from 90 to 39 and 19, respectively, after implementation of Phases One and Two.

TABLE 11. ESTIMATED ANNUAL FREQUENCY OF OVERFLOWS(1)

Item	Existing conditions	After Phase One	After Phase Two
Number of storms causing runoff	116	116	116
Number of storms causing overflow	90	39	19
Percent of total rainfall volume overflowing(1)	80	55	31

1. Includes allowance for a uniform distribution of rainfall, surface storage, surface infiltration, diversion to the treatment plant, and in-line and off-line storage.

Pollution Loads. As shown in Table 12, the reduction in total pollution discharged annually is 93 percent BOD_5 and 89 percent SS. Table 13, on the other hand, shows the reduction in pollution loads discharged during the one-year design storm.

TABLE 12. ESTIMATED TOTAL ANNUAL POLLUTION LOADS DISCHARGED

Item	Existing conditions	After Phase One	After Phase Two
BOD_5 in pounds	250,000(1)	54,000	19,000
in percent of existing conditions	-	78	93
SS in pounds	985,000(1)	310,000	110,000
in percent of existing conditions	-	68	89

1. Pollution load without diversion to treatment plant.

Note: Total annual pollution load is based on grouping storms which produce like pollution loads.

It should be noted that the overall reduction in pollution is from 90 untreated overflows per year to 19 partially treated overflows including disinfection to eliminate the chance of discharging pathogenic organisms.

TABLE 13. ESTIMATED POLLUTION LOADS DISCHARGED DURING THE ONE-YEAR DESIGN STORM

Item	Existing conditions	After Phase One	After Phase Two
BOD_5 in pounds	6,800	3,500	2,500
in percent of existing conditions	-	52	37
SS in pounds	54,300	43,100	27,000
in percent of existing conditions	-	79	50

Receiving Water Quality

The method of approach in evaluating water quality conditions in the Great Miami River resulting from combined sewer overflow remedial actions was a combination of steady state and dynamic analyses. This was done primarily to assist decision makers in evaluating how the conditions achieved fit established criteria. To date, no meaningful criteria have been set that are applicable to combined sewer overflows.

A steady-state analysis was conducted to determine the DO sag conditions in the river resulting from the Middletown loads. In this analysis, loads from combined sewer overflows were taken as the peak pollution discharges.

However, overflows are not steady continuous loads but consist of short-term slugs that flow downstream growing in extent and reducing in concentration as a result of dispersion. For this reason, another steady-state analysis was conducted in which the pollution discharge was represented as a constant discharge of the average load discharged during the peak hour of combined sewer overflow. This was judged as a condition which dispersion around the peak would cause.

Both analyses were conducted following the traditional Streeter-Phelps approach of assuming all flows and water quality conditions as not changing.

Recognizing the highly variable conditions of combined sewer overflows, dynamic analyses were also conducted. Initially, we followed the principle that storm conditions, especially during a one-year storm, are associated with increased river flows, thereby negating the use of low river flows. However, upon further evaluation, it was felt that low river flows should be used for a combined sewer overflow analysis, thereby representing a very localized storm condition.

Therefore, the dynamic analysis was conducted using low river flow equivalent to the seven-day low flows exceeded 90 percent of the time, as presented earlier. Time varying pollution loads (pollutographs), however, were used to represent the discharge loads under existing, Phase One and Phase Two conditions. The discharges from Middletown's water pollution control plant were also included as uniform discharges.

The method is a numerical network model consisting of interconnected channel reaches simulating the mass transport phenomena including advection, dispersion and biological reaction processes following first-order kinetics. The technique was developed at the Massachusetts Institute of Technology.*

The results of each analysis are shown on Figures 16 and 17 for existing conditions, Phase One and Phase Two. This analysis includes the discharge of Middletown wastes including those from its Water Pollution Control Plant, but does not consider conditions above or below the Middletown area. This analysis can be readily incorporated in a more comprehensive river basin plan.

The effects on DO from the overflow pollution slugs, as they move downstream in the dynamic analysis, are shown on Figure 17. It should be noted here that these effects are felt over only a two to three-hour period.

Superimposing the results of the dynamic analysis (see Figure 16) shows that dispersion does have a marked effect in reducing the peak DO sag as the slug moves downstream. Based on this analysis, it appears that the DO sag of a combined sewer overflow would initially follow the steady-state analysis of its peak but would recover considerably sooner due to dispersion effects.

In evaluating the criteria associated with design river flows which imply that criteria must be met 90 percent of the time, a comparison to overflow conditions can be made. As a result of Phase Two improvements, there will be about 19 overflows per year. Assuming that these will occur over a six-hour period indicates that overflows will occur 114 hours per year, or less than 10 percent of the time.

On the basis of annual pollution volumes discharged, the reduction in the volume of BOD discharged as a result of Phases One and Two is 78 and 93 percent, respectively.

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*Numerical Model for the Prediction of Transient Water Quality in Estuary Networks, October 1972, James E. Dailey and Donald R. F. Harleman, Report No. 158, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Massachusetts Institute of Technology, Cambridge, Massachusetts 02139.

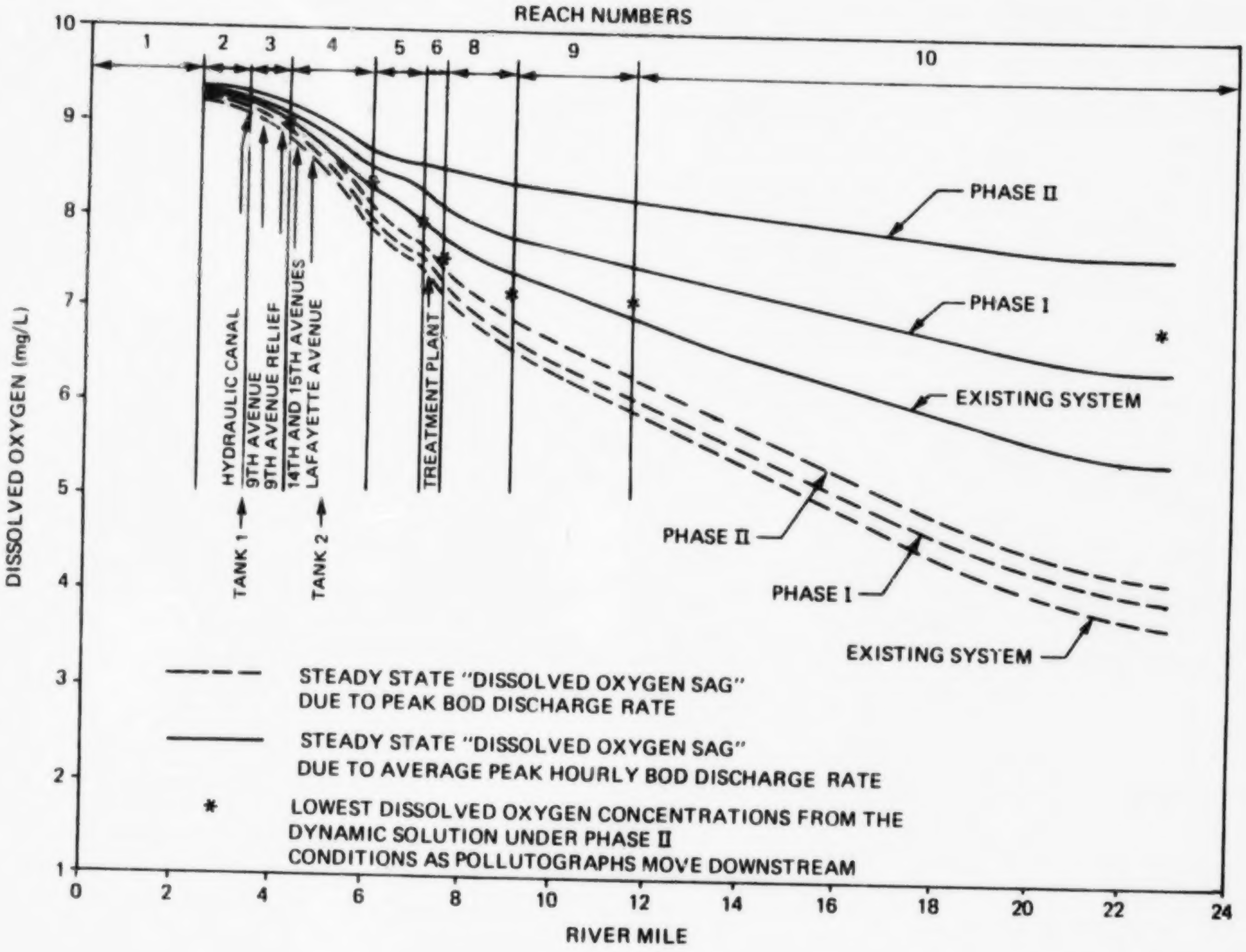

FIG. 16 DISSOLVED OXYGEN SAG - STEADY STATE AND DYNAMIC ANALYSIS FOR ONE YEAR STORM

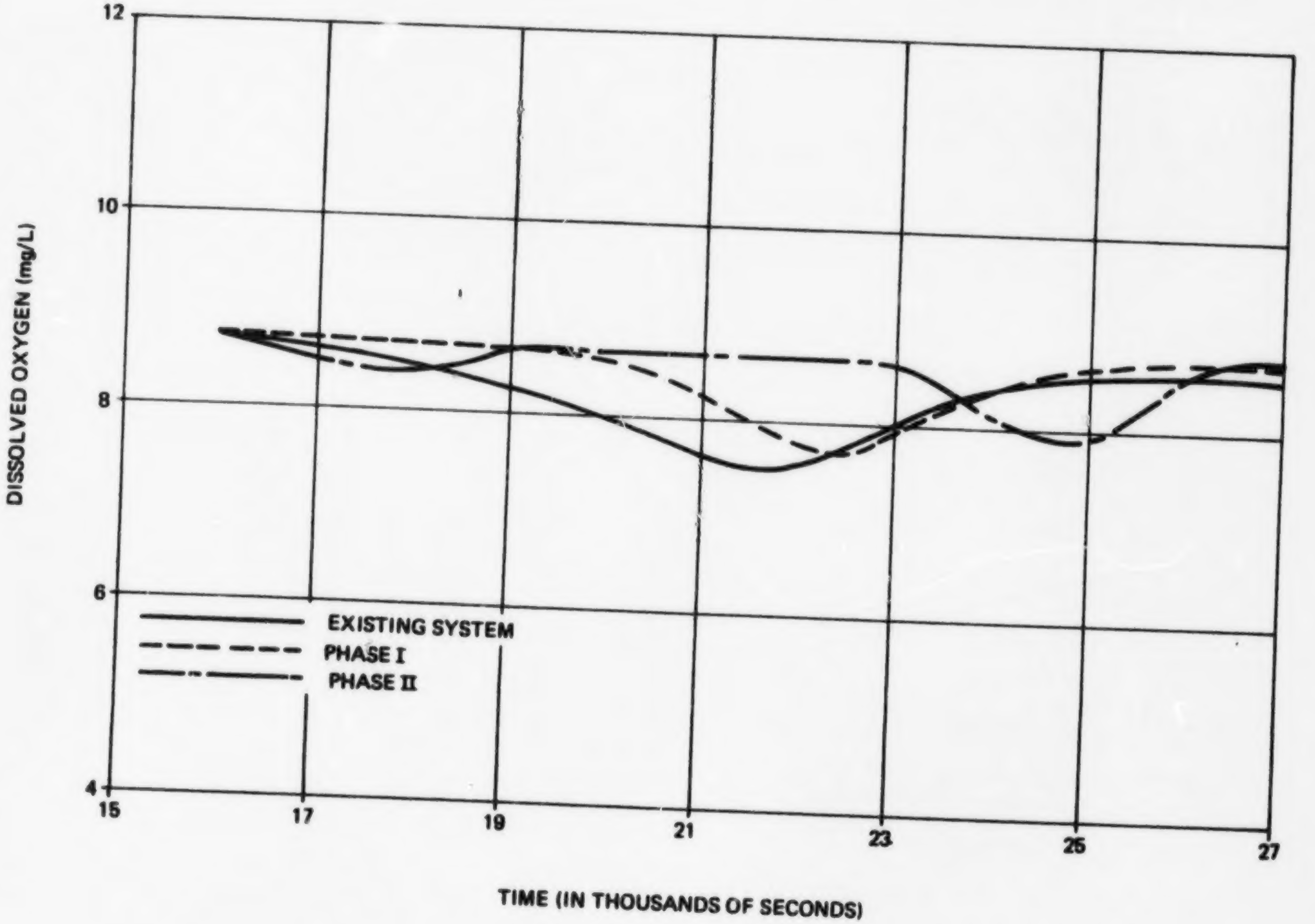

FIG. 17 DISSOLVED OXYGEN VS. TIME AT BEGINNING OF REACH NO. SIX – ONE YEAR STORM

CASE STUDY - 3 COMBINED SEWER OVERFLOW REGULATION IN THE METROPOLITAN BOSTON AREA

Introduction

Boston Harbor has three major tributary rivers, the Mystic, Charles and Neponset, which funnel through Boston and empty their accumulated flows and impurities into the Harbor. Prior to 1889, the increased expansion of local sewer systems discharging their wastes and street runoff directly into these rivers gave rise to considerable public concern. Accordingly, in 1889, the State Board of Health completed an exhaustive investigation and recommended passage of the act establishing the Metropolitan Sewer District.

Today there are 43 cities and towns in the Metropolitan Sewer District which now serves almost 2 million people from an area greater than 400 square miles.

The Metropolitan Sewer District facilities include approximately 225 miles of trunk sewers, serving nearly 5,000 miles of local sewers. The District has 11 pumping stations, four headworks, and two large primary treatment plants at Deer Island and Nut Island. These plants have an average treatment capacity of more than 450 million gallons per day, with a combined capability of handling maximum flows at the rate of 1.2 billion gallons per day.

Completing the major components of the wastewater transport and disposal system in the Metropolitan Sewer District service area are 68 major combined sewer overflows in five member communities serving an area of 36 square miles and 900,000 people. Figure 18 shows the general location of these overflows.

After having the responsibility of providing sewer service for the past 80 years, the M.D.C. is now planning for the next 80 years.

One component of this planning effort is a broad assessment of the combined sewer overflow problem in the Boston Harbor area and identification of remedial opportunities available for such pollution abatement.

During a conference on the Boston Harbor*, the biggest pollution problem confronting the Harbor was identified as the combined sewer discharge problem which has been causing bacterial contamination and visual pollution of beaches and shellfish areas.

There have been in the past, in addition to sewer separation, two general approaches to combined sewer overflow regulation in the Boston area.

1. Diversion of overflows to chlorination-detention facilities allowing for piecemeal solutions in the immediate need areas.

2. Collection and ocean discharge of overflow providing for a large-scale elimination from the Boston Harbor area.

The Cottage Farm Chlorination-Detention facility in Cambridge is an example of the first approach, and the Deep Tunnel Plan, as proposed to the City of Boston, represents the second approach.

Treatment Criteria

The suitability of the waters for bathing and other recreational activities and for the harvesting of shellfish is dependent on acceptably low pollution concentrations.

The expected future pollution control processes would be biological treatment for dry-weather flow with pretreatment of noncompatible industrial wastes. On this basis, it is expected that industrial pollutants such as toxics and heavy metals would be removed prior to entering the sewer system and would not become subject to overflow.

The critical parameter for an area's acceptability for bathing in the Boston Harbor area has been bacterial contamination. On this basis, the destruction of bacteria is set as the primary requirement of any treatment process, with reduction of solids pollution and elimination of visual pollution as additional measures of accomplishment.

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*Proceedings, Conference In the Matter of Pollution of the Navigable Waters of Boston Harbor and its Tributaries, Third Session, October 27, 1971, Boston, Massachusetts, U. S. Environmental Protection Agency.

In order to meet the bacterial standard set by the Public Health agencies, the facilities would be designed to provide 15 minutes chlorine disinfection contact time for the peak flow from a selected design storm. Other treatments accomplished with this would be: Coarse screening of influent, skimming of floatables, sedimentation, and fine screening prior to discharge.

Application of the SWMM

The SWMM is being used to model approximately 24,000 acres of combined and separate sewered areas in Boston, Brookline, Cambridge, Chelsea and Somerville. The modeling process quantifies the combined sewer overflow pollution problem in terms of quantity of discharge and amount of pollutants as represented by BOD, SS and coliform bacteria.

The overall statistics on the modeling effort are as follows:

Total population considered - 900,000
Total area modeled - 24,000 acres
No. of transport models used - 29
No. of elements modeled - 2,035
Total length of conduits modeled - 617,100 feet (7%)
No. of subareas modeled - 517
Average size of subarea - downtown -25 to 30 acres
- parks -75 to 80 acres

Model Demonstration. Although the application of the model has been demonstrated during its development and in studies thereafter, it is pertinent to test its use against any measured values in the locality of its application. Measurements taken at an overflow in Cambridge* were compared favorably with hydrographs generated by the SWMM under similar conditions.

**Combined Sewer Overflows to the Charles River*, August, 1972, Process Research, Inc., Cambridge, Mass., for the Commonwealth of Massachusetts Water Resources Commission.

Receiving Water Quality Analysis

The Massachusetts Division of Water Pollution Control in conjunction with Hydroscience, Inc., are conducting water quality modeling and simulation of quality conditions in the Harbor to determine necessary steps in achieving remedial objectives.

The model* was developed by Hydroscience, Inc., and Figure 19 shows the discretization of Boston Harbor for purposes of evaluating combined sewer overflow effects.

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*Development of Hydrodynamic and Time Variable Water Quality Models of Boston, Harbor, July 1973 Hydroscience, Inc., N. J., for the Commonwealth of Massachusetts Water Resources Commission.

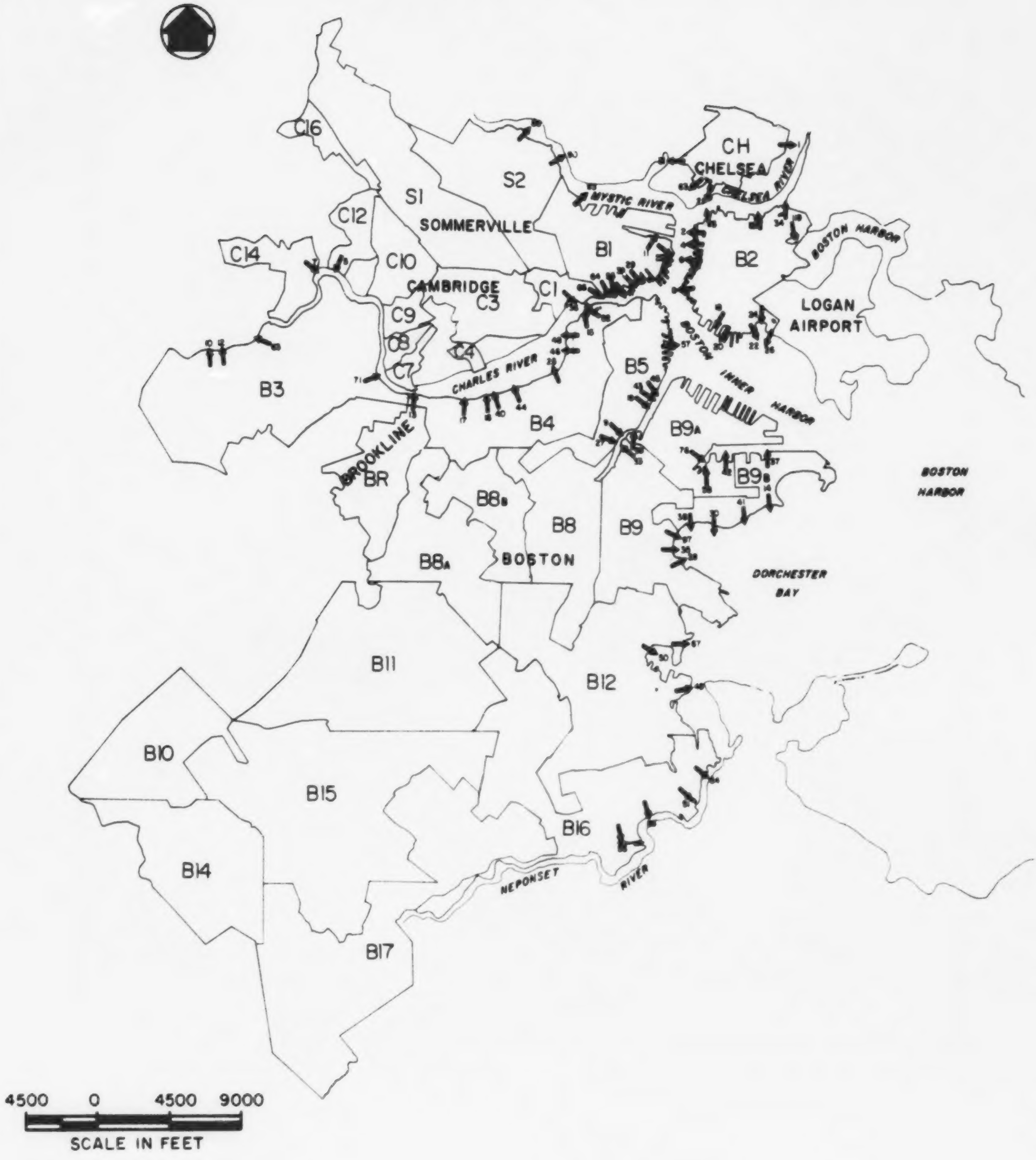
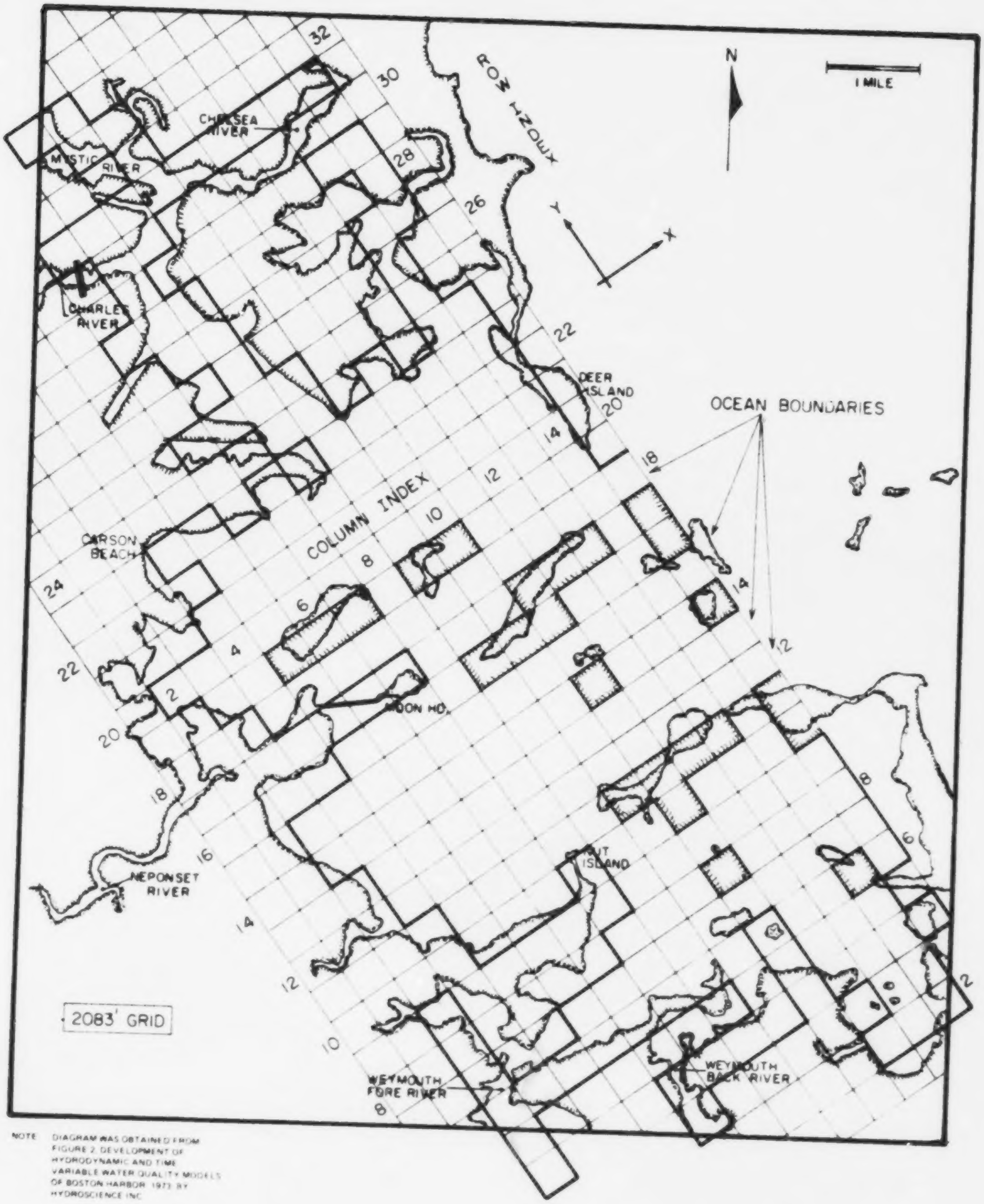

C16
CH
CHELSEA
S1
S2
MYSTIC RIVER
CHELSEA RIVER
C12
SOMMERVILLE
B1
B2
BOSTON HARBOR
C14
C10
CAMBRIDGE
C1
C3
LOGAN
AIRPORT
C9
C8
C4
BOSTON
C7
CHARLES RIVER
B5
INNER
HARBOR
B3
B4
B9A
BROOKLINE
BR
B9B
BOSTON
HARBOR
B8B
B8
B9
B8A
BOSTON
DORCHESTER
BAY
B11
B12
B10
B15
B16
B14
NEPONSET
RIVER
B17
4500
0
4500
9000
SCALE IN FEET

FIG. 18 EXISTING MAJOR OVERFLOWS EMMA AREA


N
1 MILE
Y
X
CHELSEA RIVER
MYSTIC RIVER
CHARLES RIVER
DEER ISLAND
OCEAN BOUNDARIES
COLUMN INDEX
CARSON BEACH
MOON HD.
NUT ISLAND
NEPONSET RIVER
WEYMOUTH FORE RIVER
WEYMOUTH BACK RIVER
2083' GRID
32
30
28
26
22
20
18
14
12
10
8
6
4
2
24
22
20
18
16
14
12
10
8
14
12
8
6
2

NOTE: DIAGRAM WAS OBTAINED FROM FIGURE 2, DEVELOPMENT OF HYDRODYNAMIC AND TIME VARIABLE WATER QUALITY MODELS OF BOSTON HARBOR, 1973, BY HYDROSCIENCE INC.

FIG. 19 BOSTON HARBOR HYDRODYNAMIC MODEL

COMPARATIVE ANALYSIS OF URBAN STORMWATER MODELS

By

Albin Brandstetter*

ABSTRACT

Eighteen mathematical models for the nonsteady simulation of runoff in urban storm and combined sewerage systems were reviewed in a study sponsored by the U.S. Environmental Protection Agency. The models were evaluated on the basis of information published by the model builders and model users. Seven models were also tested by computer runs using both hypothetical and real catchment data. Most of the models evaluated include the nonsteady simulation of the rainfall-runoff process and flow routing in sewers; a few also include the simulation of wastewater quality, options for dimensioning sewerage system components, and features for realtime control of overflows during rainstorms.

INTRODUCTION

In the past, the most common methods for analyzing and designing urban storm and combined sewerage systems have been based on the rational formula for rainfall-runoff computations and steady-state flow equations for sewer network flow analyses. Recognizing the limitations of these methods for highly complex systems under dynamic flow conditions, mathematical models have been developed in recent years to provide better tools for the analysis of existing and planned sewerage systems and system improvements.

In a study sponsored by the U.S. Environmental Protection Agency, mathematical models simulating dynamic wastewater flow and quality conditions for engineering assessment, control, planning and design of storm and combined sewerage systems were evaluated. The most promising models for practical application were tested using hypothetical and real urban catchment data. The evaluations considered model accuracy, the cost of model use, computer requirements, data requirements, input data preparation requirements and output options available to the user. The evaluations were intended to aid the practicing engineer in deciding which of the models will meet his requirements.

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*Research Associate, Battelle Pacific Northwest Laboratories, Richland, Washington.

Mathematical models of catchment hydrology are available which handle all or any combination of the following phenomena:

- Catchment runoff
- Flow routing
- Water quality
- Realtime control
- Design

Some of the models, however, have been developed for nonurban catchments and are not directly applicable to urban catchments without major revisions to account for special urban hydrologic phenomena such as runoff from pervious and impervious areas, flow routing in closed conduits, and wastewater quality.

Eighteen models were identified which were specifically developed or adapted for the dynamic simulation of urban catchments. All but one of these models include the nonsteady simulation of the rainfall-runoff process and all but one include flow routing in the sewers. Five models include also the continuous simulation of wastewater quality. Six models include options for dimensioning sewer pipes and three of them use mathematical optimization schemes for least-cost design of new sewer system components. Two models have provisions for the realtime control of overflows during rainstorms.

A brief review of these models indicates a tremendous diversity in scope and purpose, mathematical detail, system elements and hydrologic phenomena being modeled, size of the system that can be handled, data input requirements, and computer output. This diversity, of course, is a result of the varying conditions and objectives which govern the design and evaluation of individual sewerage systems.

The state of development of these models also varies significantly. Some models have been developed and verified extensively, others have been developed but not verified, while some have been developed conceptually but not carried to the point of application. In addition, because no standards exist for evaluating and comparing models, differing criteria have been used in the evaluations. The rigor with which the models were tested varies greatly, from intuitive judgments to graphical comparisons and more demanding statistical analyses.

The above considerations have hindered both the technology transfer to interested municipalities and consulting engineers and the routine use of these new methodologies. In fact, due to a lack of information on data input requirements, model limitations, and program output options combined with often overstated claims of capabilities, use of this advanced methodology has received some criticism.

The objective of this state-of-the-art survey is therefore to provide the engineer and planner with a readily available reference containing brief but precise descriptions and evaluations of all the models available, thus facilitating his selection of the appropriate model for a particular application.

MODEL SELECTION

This study reviewed mathematical models which can be used for the engineering assessment, planning, design, and control of storm and combined sewerage systems. Engineering assessment involves the evaluation of system performance under various hydrologic and wasteload conditions to determine problem areas and needs for system improvement. This may include the determination of undesirable surcharging, backwater and flooding conditions, the extent of overflows during rainstorms and the performance of storage facilities and treatment plants. For planning purposes, the effects of both existing and potential hydrologic conditions and land use decisions on existing and planned sewerage systems and system components would be studied.

Design involves the computation of sizes of sewers, flow control and diversion structures, and storage and treatment facilities which will meet specified performance criteria. Design considerations include the elimination of undesirable flooding, the reduction of untreated overflows, and better use of existing and planned facilities.

Control involves the regulation of flows to improve system efficiency. It may include the regulation of controllable diversion structures and the operation of storage reservoirs and treatment plants during rainstorms to maximize the utilization of available system capacity and minimize untreated discharges to receiving waters. The need for potential system improvements and the implementation of realtime control schemes for manual or fully automatic control of wastewater flows, storage and treatment may also be considered in this phase.

To serve the purposes described above, mathematical models must consider the spatial nonuniformity of rainfall; the time-varying runoff resulting from rainstorms of different intensities and durations; spatial and temporal variations in dry-weather flows; the effect on hydrograph shapes of different flow travel times from various catchments; the attenuation of flood peaks during overland, gutter, and sewer conduit flow routing; and the operation of flow diversion structures and storage facilities under dynamic wasteload conditions. The models should be able to combine the runoff from several catchments and route the wastewaters within the sewer networks.

Most of the 18 selected models meet the above requirements. The few exceptions included are models with special provisions or advantages for the evaluation of important aspects of urban catchments and sewerage systems, or with features which could improve existing models or provide new modeling capability.

All 18 models were evaluated on the basis of published information and communication with model developers. In addition, seven of the more comprehensive models were selected for testing by computer runs using hypothetical and real catchment data.

In selecting these models, a minimum requirement was the capability to consider several raingauges, to compute runoff from several catchments, and to route flows in a converging branch sewer network. Models which rely heavily on mathematical formulations which cannot be derived readily from catchment and sewer physical characteristics were not tested, nor were models whose oversimplification restricts their use unnecessarily, considering present computer capabilities and the state-of-the-art of hydrologic modeling.

The 11 models which were evaluated solely on the basis of published information in reports by model builders and model users are:

1) British Road Research Laboratory (BRRL) Model

2) Chicago Hydrograph Method

3) Colorado State University Urban Runoff Modeling

4) Corps of Engineers Hydrologic Engineering Center Storage, Treatment, Overflow and Runoff Model

5) Hydrocomp Simulation Program

6) Minneapolis-St. Paul Urban Runoff Model

7) Seattle Computer Augmented Treatment and Disposal System

8) University of Cincinnati Urban Runoff Model

9) University of Illinois Storm Sewer System Simulation Model

10) University of Massachusetts Combined Sewer Control Simulation Model

11) Wilsey & Ham Urban Watershed System

The seven models which were also tested by computer runs are:

1) Battelle Urban Wastewater Management Model

2) Dorsch Consult Hydrograph-Volume Method

3) Environmental Protection Agency Storm Water Management Model (SWMM)

4) Massachusetts Institute of Technology Urban Watershed Model

5) Metropolitan Sanitary District of Greater Chicago Flow Simulation Program

6) SOGREAH Looped Sewer Model

7) Water Resources Engineers Storm Water Management Model

A review of all 18 models and preliminary results of the numerical testing of four models is presented in the following sections.

MODEL REVIEWS

The two types of tables presented in the following pages provide a quick overview of the features of each model. The first type, Table 1, lists major model features and indicates whether a certain phenomenon is being modeled or considered by a model. The table consequently indicates the comprehensiveness of each model. The second type, Tables 2 through 5, expands on Table 1 by listing additional categories, brief statements indicating the mathematical formulations for particular phenomena, model limitations, and other details which would be helpful in assessing the applicability of a model for a particular purpose.

It is impossible to describe all the features of each model in a state-of-the-art review and to discuss every model advantage and limitation based solely on information provided by the model builders and selected numerical tests. This would be possible perhaps for evaluations of mathematical models of individual pnenomena, but it becomes a tremendous task when the interactions of all the submodels of the comprehensive models in this study must be considered. Only extensive use and experimentation with a particular model will reveal all its advantages and shortcomings. Most of the models are being revised, expanded, and improved continuously by the model developers. New versions of some models are being developed by various model users.

It is hoped, however, that this review will provide sufficient information to the potential user to aid him in selecting the appropriate model for his needs. Once the user has decided that a model may be of interest to him, careful study of the documents referenced for the model is recommended before its computer implementation. This would familiarize him with some model details which may be important to him but cannot be described sufficiently in this review.

The model comparisons emphasize evaluation of the complete model, rather than detailed critiques of submodels of individual phenomena. Formulations of individual phenomena are discussed only in the context of their use in the complete model, and no attempt is made to discuss all available models for each phenomenon. As an example, a tremendous variety of models exist for simulating the rainfall-runoff process alone. No attempt was made to list all of them. Rather, only those used in the

Table 1. COMPARISON OF MAJOR MODEL CATEGORIES

	CATCHMENT HYDROLOGY							SEWER HYDRAULICS								WASTEWATER QUALITY									MISCELLANEOUS				
	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	INPUT OF SEVERAL HYETOGRAPHS	SNOWMELT	RUNOFF FROM IMPERVIOUS AREAS	RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS	FLOW ROUTING IN SEWERS	UPSTREAM AND DOWNSTREAM FLOW CONTROL	SURCHARGING AND PRESSURE FLOW	DIVERSIONS	PUMPING STATIONS	STORAGE	PRINTS STAGE	PRINTS VELOCITIES	DRY-WEATHER QUALITY	STORMWATER QUALITY	QUALITY ROUTING	SEDIMENTATION AND SCOUR	QUALITY REACTIONS	WASTEWATER TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION	CONTINUOUS SIMULATION	CAN CHOOSE TIME INTERVAL	DESIGN COMPUTATIONS	REAL-TIME CONTROL	COMPUTER PROGRAM AVAILABLE
BATTELLE NORTHWEST	●	●	●		●	●		●			●		●	●	●	●	●	●			●					●	●	●	●
BRITISH ROAD RESEARCH LABORATORY	●	●	●		●			●															●			●			●
CHICAGO FLOW SIMULATION	●	●	●	●	●	●	●	●					●	●									●		●	●			●
CHICAGO HYDRO-GRAPH METHOD	●	●	●		●	●	●	●															●			●	●		●
COLORADO STATE UNIVERSITY	●	●			●			●	●					●	●														●
CORPS OF ENGINEERS				●	●	●	●				●		●				●			●		●			●				●
DORSCH CONSULT	●	●	●		●	●		●	●		●		●	●	●								●			●			
ENVIRONMENTAL PROTECTION AGENCY	●	●	●		●	●		●	●	●	●	●	●			●	●	●	●	●	●		●	●		●	●		●
HYDROCOMP	●	●	●	●	●	●	●	●	●		●		●	●	●	●	●	●		●		●	●	●	●	●		●	
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	●	●	●		●	●		●						●	●								●			●	●		
MINNEAPOLIS-ST. PAUL	●	●	●		●	●		●			●															●		●	●
SEATTLE	●	●	●		●	●	●	●	●	●	●	●	●	●												●		●	●
SOGREAH	●	●			●	●		●	●	●	●	●	●	●	●			●					●	●		●			
UNIVERSITY OF CINCINNATI	●				●	●		●															●			●			●
UNIVERSITY OF ILLINOIS	●	●		●	●	●		●	●		●	●	●	●	●											●	●		●
UNIVERSITY OF MASSACHUSETTS	●	●			●		●	●	●					●	●														●
WATER RESOURCES ENGINEERS	●	●	●		●	●		●	●	●	●	●	●	●	●	●	●	●		●			●	●		●			
WILSEY AND HAM	●				●	●		●																		●	●		

Table 2. COMPARISON OF HYDROLOGIC FEATURES OF MODELS

	Multiple catchment inflows	Dry-weather flow	Subcatchment precipitation	Evaporation	Snow accumulation and melt	Frozen ground	Infiltration on impervious areas
Battelle-Northwest	Yes	Hourly, daily, and seasonal patterns*	Weighted avg. of several raingages for each subcatchment	No	No	No	Exponential decay function, dependent on catchment moisture
British Road Research Laboratory	Yes	Straight line*	Weighted avg. of several raingages for each subcatchment	No	No	No	No
Chicago Flow Simulation	Yes	Hourly flow*	Not more than one per subcatchment	Nonlinear function of soil moisture and daily avg. air temperature	Based on Bureau of Reclamation Engineering Monograph 35 method	No	No
Chicago Hydrograph Method	Yes	Hydrographs*	Weighted avg. of several raingages for each subcatchment	Fixed function, independent of meteorological variations	No	No	No
Colorado State University	Yes, but only into single pipe string	Hydrographs*	No	No	No	No	No
Corps of Engineers	For one catchment only	No	One raingage for single catchment	Constant rate* between rainstorms for each month	Degree-day method	No	Constant fraction of rainfall minus depression storage
Dorsch Consult	Yes	Diurnal pattern computed from land use	One raingage per subcatchment, special statistical analyses	No	No	No	No
Environmental Protection Agency	Yes	Hourly, daily, and seasonal patterns* or computed from land use	One raingage per subcatchment	No	No	No	No
Hydrocomp	Yes	Hydrographs*	Percent of gage precip., one per subcatchment	Based on measured potential evaporation and available moisture	Based on Corps of Engineers method	Heat exchange equ.	No
Massachusetts Institute of Technology	Yes	Hydrographs*	One raingage per subcatchment, storm can be moved across catchment	Penman equ.	No	No	No
Minneapolis-St. Paul	Yes	Diurnal pattern*	Weighted avg. of several raingages for each subcatchment	No	No	No	Exponential decay function
Seattle	Each catchment represented independently	Diurnal pattern*	Six raingages	No	No	No	Empirical functions**
Sogreah	Yes	Hydrographs*	Synthetic design storm computed from subcatchment characteristics	No	No	No	Synthetic rainfall excess
University of Cincinnati	Yes	No	One raingage for entire sewerage system	No	No	No	No
University of Illinois	Yes	Hydrographs*	Yes**	No	Based on Corps of Engineers method	No	Horton equ.
University of Massachusetts	Yes, but only into single pipe string	Constant flow*	One raingage for entire sewerage system, generation of syn. rainfall	No	No	No	No
Water Resources Engineers	Yes	Avg. flow* transformed into hourly and daily patterns based on land use	Up to three raingages per subcatchment	No	No	No	Constant rate*
Wilsey and Ham	Yes	No	One raingage for entire sewerage system	No	No	No	

Notes: Blank space indicates that relevant information is not available.
*Provided as input data.
**Mathematical formulation and/or method of solution not available.

Table 2 (Continued)

Other rainfall losses on impervious areas	Storm runoff from impervious areas	Infiltration on pervious areas	Other rainfall losses on pervious areas	Storm runoff from pervious areas	Groundwater simulation	Gutter flow	Water balance between storms
Initial loss* before runoff begins	Unit hydrograph	Modified Holtan equ., dependent on soil moisture	Initial loss* before runoff begins	Synthetic unit hydrograph	No	No	No
Loss as function of time*	Rainfall excess routed using time-area curve	No	No	No	No	Avg. travel time from Manning equ.	No
No	Constant fraction of overland flow storage	Polynomial regression equ., dependent on soil moisture	No	Linear storage routing of overland flow storage	No	No	Accounting of soil moisture, overland and channel storage
Initial loss* before runoff begins, recovery between storms	Izzard's equ.	Modified Horton's equ., dependent on soil moisture	Function of moisture conditions, recovery between storms	Izzard's equ.	No	Kinematic wave equ., with Manning equ.	Accounting of soil moisture, depression, overland and channel storage
No	Hydrographs*	No	No	Hydrographs*	No	No	No
Constant loss rate* modified by evaporation	Constant fraction of rainfall minus depression storage	Constant fraction of rainfall minus depression storage	Constant loss rate* modified by evaporation	Constant fraction of rainfall minus depression storage	No	No	Accounting of depression and reservoir storage
No depression storage on roofs, initial loss on streets before runoff begins	Kinematic wave equ.	Option to use Horton or Holtan equ.	Function of moisture conditions	Kinematic wave equ.	No	Computed as surface runoff or channel routing	Recovery of depression storage on pervious areas
Initial loss* before runoff begins	Kinematic wave equ., with Manning equ.	Horton equ., independent of soil moisture	Initial loss* before runoff begins	Kinematic wave equ., with Manning equ.	No	Kinematic wave equ., with Manning equ.	No
Initial loss* modified by evaporation before runoff begins	Kinematic wave equ., with Manning equ.	Empirical equ., dependent on soil moisture	Initial loss* modified by evaporation before runoff begins	Kinematic wave equ., with Manning equ.	Nonlinear functions for storage and surface flow contribution	No	Accounting of soil moisture, groundwater and surface water storage
	Kinematic wave equ., with Manning equ.	Opt. to use Horton equ., Holtan equ., SCS method, runoff coefficient method		Kinematic wave equ., with Manning equ.	No	Kinematic wave equ., with Manning equ.	No
Initial loss* before runoff begins	Unit hydrograph	Modified Holtan equ., dependent on soil moisture	Initial loss* before runoff begins	Unit hydrograph	No	No	No
No	Unit hydrograph	Empirical functions**	No	Unit hydrograph	No	No	Runoff adjusted based on previous storm**
Synthetic rainfall excess	Muskingum method	Synthetic rainfall excess	Synthetic rainfall excess	Muskingum method	No	No	No
Depression loss function of rainfall	Kinematic wave equ., with Manning equ.	Horton equ., with time offset based on actual rainfall, indep. of soil moisture	Depression loss function of rainfall and infiltration	Storage routing with Manning equ.	No	Outflow equals inflow during same time interval	No
Yes**	Kinematic wave equ., with Manning equ.	Horton equ.	Yes**	Kinematic wave equ., with Manning equ.	No	Kinematic wave equ., with Manning equ.	No
Initial loss* before runoff begins	Constant fraction of rainfall minus initial loss	No	No	No	No	No	Recovery of depression storage
Initial loss* before runoff begins	Kinematic wave equ., with Manning equ.	Horton equ.	Initial loss* before runoff begins	Kinematic wave equ., with Manning equ.	No	Kinematic wave equ., with Manning equ.	No
Yes**	Kinematic wave equ., with Manning equ.	Yes**	Yes**	Kinematic wave equ., with Manning equ.	No	Kinematic wave equ., with Manning equ.	No

Table 3. COMPARISON OF HYDRAULIC FEATURES OF MODELS

	Open channel network	Free surface flow	Backwater effects	Flow reversal	Surcharging and pressure flow	Different pipe cross-sections	Different diversion structures
Battelle Northwest	Converging branch network	Kinematic wave equ., characteristic solution, Manning equ.	No	No	Considered independently in each pipe	Circular pipe	Four common types of orifice/weir combinations
British Road Research Laboratory	Converging branch network	Storage routing, Manning equ.	No	No	No	Circular and rectangular pipe, trapezoidal channel	No
Chicago Flow Simulation	Converging branch network	Storage routing, Manning equ.	No	No	No	Circular pipe, trapezoidal channel with flood plain	No
Chicago Hydrograph Method	Converging branch network	Storage routing, Manning equ.	No	No	No	Circular pipe, trapezoidal channel	No
Colorado State University	Single string of pipes	Dynamic wave equ., various solutions, Manning or Darcy-Weisbach equ.	Dynamic wave equ.	Dynamic wave equ.	No	Circular pipe	No
Corps of Engineers	No flow routing	No	No	No	No	No	Undefined shape
Dorsch Consult	Converging and diverging branch and loop network without flow reversal	Dynamic wave equ., implicit finite diff. sol., Manning or Prandtl equ.	Dynamic wave equ.	No	Yes**	Various specified shapes plus arbitrary shape	Weir, diverging pipe branches
Environmental Protection Agency	Converging branch network	Kinematic wave equ., explicit finite diff. sol., Manning equ.	For known depth vs. storage relationship at not more than two locations	No	Considered independently in each pipe	Twelve specified shapes plus three arbitrary shapes	Two types: unspecified shape or weir
Hydrocomp	Converging branch network	Kinematic wave equ., with diffusion term,* Manning equ.	Diffusion term in kinematic wave equ.	No	No	Circular pipe, trapezoidal channel with flood plain	Diversion hydrographs*
Massachusetts Institute of Technology	Converging branch network	Kinematic wave equ., explicit or implicit finite diff. sol., Manning equ.	No	No	No	Circular pipe, triangular gutter, triangular and rectangular channel	No
Minneapolis-St. Paul	Converging branch network	Progressive avg. lag method	No	No	No	Circular pipe	Three types of common orifice/weir comb., three special structures
Seattle	Converging branch network	Dynamic wave equ., explicit finite diff. sol., Manning equ.	Dynamic wave equ.	No	Considered only for trunk storage upstream of regulators	Circular and horseshoe pipe	Six types of gate control structures
Sogreah	Converging and diverging branch and loop network	Dynamic wave equ., implicit finite diff. sol., Manning equ.	Dynamic wave equ.	Dynamic wave equ.	Yes**	Circular and egg-shaped pipe, trapezoidal channel plus arbitrary shapes	Rectangular weir and orifice, including time-varying gate settings
University of Cincinnati	Converging branch network	Translation of hydrograph by avg. flow time of hydrograph, Manning equ.	No	No	No	Circular pipe and rectangular channel	No
University of Illinois	Converging branch network	Dyn. wave equ., explicit finite diff. sol. of char. equ.'s, Darcy-Weisbach	Dynamic wave equ.	Dynamic wave equ.	No	Circular pipe	Weir or gate flow control and diversion at outlet
University of Massachusetts	Single string of pipes	Dynamic wave equ., implicit finite diff. sol., Manning equ.	Yes		No	Circular pipe	No
Water Resources Engineers	Converging and diverging branch and loop network	Dynamic wave equ., explicit finite diff. sol., Manning equ.	Dynamic wave equ.	Dynamic wave equ.	Considered independently at each junction**	Circ., rect., horseshoe, basket handle and egg-shaped pipe, trap. channel	Weirs, gates or orifices, with or without tide gates
Wilsey and Ham	Converging branch network	Kinematic wave equ.,** Manning equ.	No	No	No		No

Notes: Blank space indicates that relevant information is not available.
*Provided as input data.
**Mathematical formulation and/or method of solution not available.

Table 3 (Continued)

Diversion computation	Pumping stations	Overflow storage	Inline storage	Storage computation	Infiltration into sewers or open channels	Computes stage hydrographs	Computes flow velocities
Orifice and weir eqs. neglecting downstream conditions	No	For undefined shape, only inflow and volume computed	No	Computes inflow and volume of overflow storage	No	For each pipe reach	For each pipe reach
No	No	No	No	No	No	No	No
No	No	No	For undefined shape, no depth computations	Stores inflow exceeding specified max. outflow	No	For each pipe and channel reach	No
No	No	No	No	No	No	No	Yes, but not printed
No	No	No	No	No	No	At any point	At any point
Excess over storage and treatment capacity overflows	No	Excess over treatment capacity is stored	Excess over treatment capacity is stored	Stored water returns to treatment plant	No	No	No
Weir eqs., considering upstream and downstream conditions	No	Yes	Yes	Function of shape, inflow and outflow	Constant rate*	For each sewer system element	For each sewer system element, but prints only selected values
Constant diversion or weir eqs. neglecting downstream conditions	Constant discharge if wet well exceeds specified depth	One only, models four types of outlet structures	Two only, models four types of outlet structures	Function of shape, inflow and outflow	Constant rate*	Yes, but not printed	Yes, but not printed
Diversion hydrographs*	Pumping schedule*	Yes**	Yes, considers both spillway and power turbine flow	Rule curve operation or preset outflows (including power flows)	Nonlinear function dependent on soil moisture	For each channel reach and storage	For each pipe and channel reach and storage
No	No	No			No	For each pipe and channel reach	For each pipe and channel reach
Orifice and weir eqs.'s neglecting downstream conditions	No	No	No	*	No	No	No
Orifice and weir eqs.'s considering upstream and downstream conditions	Two types: proportional and constant rate	No	Storage in trunk sewers upstream of regulators	Function of shape, inflow and outflow	Decay function of storm	For each inflow point and junction	For each pipe reach
Orifice and weir eqs.'s considering upstream and downstream cond. & flow reversal	Yes**	Yes	Yes	Yes**	Hydrographs*	For each sewer system element & computational point	For each sewer system element & computational point
No	No	No	No	No	No	For each pipe reach, but not printed	For each pipe reach, but not printed
Weir eqs. or any discharge vs. depth function	Pumping rate can be outlet boundary condition	No	Yes	Function of shape, inflow and outflow	No	For each sewer system element and at any desired points	For each sewer system element and at any desired points
No	No	No	No	No	No	For each pipe reach	For each pipe reach
Orifice and weir eqs.'s considering upstream and downstream cond. & flow reversal	Up to 3-stage pumping stations, rule curve operation	No	Yes	Function of shape, inflow and outflow	Constant rate*	For each sewer junction	For each pipe and channel reach
No	No	No	No	No	No	Yes	Yes

Table 4. COMPARISON OF WATER QUALITY FEATURES OF MODELS

	Quality constituents	Dry-weather quality	Storm runoff quality	Quality interactions on catchments	Quality routing in channels	Sedimentation and scour in channels	Quality reactions in channels
Battelle Northwest	7 arbitrary conservative constituents	Hourly, daily and seasonal patterns*	Linear regression eqs., with storm discharge and volume	No	Pure advection, mixing between successive time steps	No	No
British Road Research Laboratory	No	No	No	No	No	No	No
Chicago Flow Simulation	No	No	No	No	No	No	No
Chicago Hydrograph Method	No	No	No	No	No	No	No
Colorado State University	No	No	No	No	No	No	No
Corps of Engineers	Suspended and settleable solids, BOD, nitrogen, and phosphate	No	Nonlinear funct. of catchment char., poll. accumulation, and runoff	BOD, nitrogen and phosphate dep. on suspended and settleable solids	No	No, but land surface erosion by universal soil loss eqs.	No
Dorsch Consult	Separate model in preparation	No	No	No	No	No	No
Environmental Protection Agency	Suspended and settleable solids, BOD, COD, coliform, N, PO_4, oil and grease	Hourly, daily and seasonal patterns* or computed from land use	Nonlinear funct. of catchment char., poll. accumulation, and runoff	BOD, nitrogen and phosphate dep. on suspended and settleable solids	Pure advection, mixing between successive time steps	Suspended solids, considering particle size distribution	First-order decay without interactions for BOD
Hydrocomp	17 constituents including water temperature	Inflow concentrations* or nonlinear function of inflow hydrograph	Nonlinear funct. of catchment char., poll. accumulation, and runoff	No	Pure advection, weighted mixing between successive time steps	No	Various reactions and interactions
Massachusetts Institute of Technology	No	No	No	No	No	No	No
Minneapolis-St. Paul	No	No	No	No	No	No	No
Seattle	No	No	No	No	No	No	No
Sogreah	Conservative constituents	No	No	No	Pure advection	No	No
University of Cincinnati	No	No	No	No	No	No	No
University of Illinois	No	No	No	No	No	No	No
University of Massachusetts	No	No	No	No	No	No	No
Water Resources Engineers	Suspended and settleable solids, BOD, N, PO_4, oil and grease, 17 cons. const.	Avg. quality* transformed into hourly and daily patterns based on land use	Nonlinear funct. of catchment char., poll. accumulation, and runoff	All constituents dependent on suspended and settleable solids	Pure advection	No	No
Wilsey and Ham	No	No	No	No	No	No	No

Notes: Blank space indicates that relevant information is not available.
*Provided as input data.
**Mathematical formulation and/or method of solution not available.

Table 4 (Continued)

Quality routing through storage	Quality reactions in storage	Overflow treatment	Inline treatment	Quality interactions during treatment	Quality balance between storms	Receiving water flow simulation	Receiving water quality simulation
No	No	Tabular function of flow and concentration	Tabular function of flow and concentration	No	No	No	No
No	No	No	No	No	No	Yes	No
No	No	No	No	No	No	Yes	No
No	No	No	No	No	No	Yes	No
No	No	No	No	No	No	No	No
Plug flow	No	Computes only influent flow and pollutants	Computes only influent flow and pollutants	No	Function of pollutant accumulation and street sweeping	No	No
No	No	No	No	No	No	Yes	Separate model in preparation
Plug flow or instantaneous mixing	No	One facility only, eqn.'s for nine physical/chemical processes	No	Coliform removal dependent on suspended solids	No	For one and two-dimensional representation	For one and two-dimensional representation
Weighted mixing between one to nine horizontal layers, outflow from any layer	Various reactions and interactions**	No	No	No	Function of pollutant accumulation and street sweeping	Yes	Yes
No	No	No	No	No	No	Yes	No
No	No	No	No	No	No	No	No
No	No	No	No	No	No	No	No
Instantaneous mixing	No	No	No	No	No	Yes	No
No	No	No	No	No	No	Yes	No
No	No	No	No	No	No	No	No
No	No	No	No	No	No	No	No
Exponential release function for catchbasins	No	No	No	No	No	Yes	Yes
No	No	No	No	No	No	No	No

Table 5. COMPARISON OF MISCELLANEOUS FEATURES OF MODELS

	Continuous simulation	Time interval	Min. and max. time period for simulation	Allows input of initial conditions	Design computations	Least-cost design computations	Realtime control optimization
Battelle-Northwest	Single storm period, no water and quality balance between storms	Constant intervals in minutes (min. = 2, max. = 60)	Min. 1 hr, max. 6 hrs or 56 time steps	Catchment moisture cond., init. cond. not needed for routing	Least-cost sizes of sewers, storage and treatment facilities	Modified gradient technique considering flow and quality	Dynamic programming for regulator operation
British Road Research Laboratory	Single storm period, no water balance between storms	Constant intervals in minutes	No apparent min., max. 500 time steps	Catchment moisture conditions, not for channels	No	No	No
Chicago Flow Simulation	Continuous	Constant intervals in minutes (min. = 5, max. = 60)	Min. 24 hours, no apparent max.	No	No	No	No
Chicago Hydrograph Method	Single storm period, water balance between storms	Constant intervals in minutes (1 minute req'd for rain, can be more for runoff)	No apparent min., max. 500 minutes	No	Sizes pipes for peak flow, no cost computations	No	No
Colorado State University	Continuous flow routing in single string of pipes	Dependent on numerical stability condition and selected distance interval	No apparent min. or max.	Constant discharge in entire pipe string	No	No	Different schemes were investigated
Corps of Engineers	Continuous for several years	Must be 1 hour	Min. 24 hours, no apparent max.	Catchment moisture and quality cond., not for channels and storage	No	No	No
Dorsch Consult	Single storm period, no water balance between storms	Constant interval in minutes, restricted by numerical stability condition	No apparent min., max. 500 time steps	Catchment moisture conditions, not needed for routing	No	No	No
Environmental Protection Agency	Single storm period, no water and quality balance between storms	Constant intervals in minutes	No apparent min., max. 150 time steps	Catchment moisture and quality cond., not for channels and storage	Costs of storage and treatment, sizes pipes to eliminate surcharging	No	No
Hydrocomp	Continuous for several years	Minutes, hourly, bi-daily, daily or semi-mo.; compu. use om. intervls	No apparent min. or max.	For all sewer system elements	No	No	Flow forecasting and system operation without optimization
Massachusetts Institute of Technology	Continuous, except for snow	Constant interval in minutes		Catchment moisture conditions, not for channels	Least-cost sizes of pipes, storage and treatment facilities	Linear programming considering flow only	No
Minneapolis-St. Paul	Single storm period, no water balance between storms	Constant interval in seconds	No apparent min. or max.	Catchment moisture conditions, not for channels	No	No	Iterative runs with trial regulator settings
Seattle	Single storm period, limited water balance between storms	Constant interval in minutes	Min. 3 hours, max. 24 hours	No	No	No	Autom. regulator operation, using rule curves, each operated independently
Sogreah	Single storm period, no water balance between storms	Constant interval in minutes, can be diff. for each hour	Min. 1 hour, no apparent max.	For each sewer system element	No	No	No
University of Cincinnati	Single storm period, no water balance between storms	Constant interval in minutes	No apparent min., max. 120 time steps	Catchment moisture conditions, not for channels	No	No	No
University of Illinois	Single storm period unless continuous hydrographs are input	Constant interval in seconds, restricted by numerical stability condition	No apparent min. or max.	Constant flow	Sizes of pipes for peak flow, or max. depth no cost computations	Dynamic programming with cost and risk consideration in preparation	No
University of Massachusetts	Continuous except for snow	Must be 1 hour	Min. 1 hour, max. 1 month		No	No	No
Water Resources Engineers	Single storm period, no water and quality balance between storms	Constant interval, restricted by numerical stability condition	No min. or max.	For all sewer system elements	No	No	No
Wilsey and Ham	Single storm period, no water balance between storms			Catchment moisture conditions, not for channels	Sizes of pipes for peak flow	No	No

Notes: Blank space indicates that relevant information is not available.
*Provided as input data.
**Mathematical formulation and/or method of solution not available.

Table 5 (Continued)

Error messages	Principal printed output	Principal graphic output	Tested on urban data	Units of measurement	Computer versions and core storage requirement	Computer language	Computer program available
Limited	Rain, depth, vol., disch., conc., qual. costs and sizes of sewers, stor. and treatment	Network diagram, rain, discharge, storage, quality	Limited	British, concentrations metric*	Simulation: DEC PDP-9, 32K bytes; optimization: UNIVAC 1108, 64K bytes	Simulation: Fortran IV, 5000 statements; optim. Fortran V, 2300 statements	Yes, draft documentation
	Discharge	Discharge	Extensive	British	IBM 360	Fortran IV, 1300 statements	Yes
Limited	Rain, snow, depth, discharge, storage	Depth, discharge, storage	Limited	British	IBM 360, CDC 6400, 32K bytes	Fortran IV, 650 statements	Yes, draft user's manual
Limited	Discharge, sewer diameters	Discharge	Limited	British	IBM 1130 8K bytes	Fortran IV	Yes, draft user's manual
Limited	Depth, velocity, discharge	No	Tested on experimental pipe string (91 cm = 3 ft. diameter)	British	CDC 6400 and 6600	Fortran IV	Yes
Moderate	Rain, discharge, storage, quality, land erosion	Storage utilization	Limited	British, concentrations metric	UNIVAC 1108, IBM 360/50, CDC 6600, 70K bytes	Fortran IV, 4400 statements	Yes, draft user's manual
Extensive, plus separate data checking program (250 messages)	Depth, discharge, (separately for sanitary and storm water)	Discharge	Moderate	Metric and British	UNIVAC 1108, CDC 6600, IBM 360/50	Fortran IV, 4000 statements	Under use agreement
Moderate	Rain, discharge, quality, costs of storage and treatment, sewer sizes	Rain, discharge, quality	Moderate	British, concentrations metric	IBM 360 and 370, CDC 6600, UNIVAC 1108, 350K bytes	Fortran IV, more than 10,000 statements	Yes, being expanded with new model features
Extensive	Rain, snow, soil moisture, depth, discharge, storage, quality	Discharge, quality	Moderate	Metric and British	IBM 360 and 370, 240K bytes	PL/1	Under use agreement
Moderate	Rain, depth, discharge; costs and sizes of sewers, storage and treatment	Network diagram, rain, discharge	Limited	British	IBM 360/67		No
Limited	Rain, discharge, catchment moisture conditions, regulator status	No	Limited	British	DEC PDP-9, 32K bytes	Fortran IV	Yes
Adequate for real-time overflow control	Rain, regulator flow depth and control status	No	Limited	British	Xerox SIGMA 2, 32K bytes	Fortran IV	Yes, but real-time control functions are system specific
Moderate	Depth, velocity, discharge, pressure, street flooding depth	Depth, discharge	No	Metric	IBM 360/65, 65K bytes for 1000 computational points	Fortran IV, 3000 statements	No
None	Discharge	No	Limited	British	IBM 360/65	Fortran IV, 850 statements	Yes
Moderate	Depth, velocity, discharge, sewer diameters	Depth, discharge	Limited	British	IBM 360/75, 100 to 300K bytes	PL/1 and assembler language, 3000 statements	Yes
	Depth, discharge	No	No	British	CDC 3600	Fortran IV	
Moderate	Depth, velocity, discharge, quality	Depth, discharge	Moderate	Metric and British	UNIVAC 1108, IBM 360/65, CDC 6600	Fortran IV	Under use agreement
	Discharge, sewer diameters			British	CDC 6600, Tymshare XDS940		Under use agreement

* Equivalent to U.S. Customery Units

comprehensive urban hydrologic models were reviewed. This type of evaluation provides the engineer with the information he needs to select a comprehensive model, but does not require him to be familiar with the many available variations for modeling each urban hydrologic phenomenon.

Battelle Urban Wastewater Management Model

The Battelle Urban Wastewater Management Model is intended primarily for the simulation of large urban catchments. It simulates the time-varying runoff and water quality in combined sewerage systems consisting of several catchments and a converging branch sewer network (Brandstetter, et al., 1973). Up to seven conservative water quality constituents can be modeled. The model can be used for realtime control of overflows during rainstorms and for least-cost design of sewerage system modifications. The model is limited to the simulation of single runoff events.

Rainfall for each catchment is computed as the weighted average of rain at several raingauges. Losses are subtracted from rainfall for both pervious and impervious areas. A modified Holtan's equation is used to compute infiltration on pervious areas accounting for changes in soil moisture conditions. Runoff from rainfall excess is computed using synthetic unit hydrographs derived from physical catchment characteristics by a method of the U.S. Soil Conservation Service.

Stormwater quality is computed from stormwater runoff using regression equations including seasonal adjustment factors. Dry-weather flow and quality is provided as input data in the form of average values for each catchment and adjustment factors for diurnal, weekday and seasonal variations.

The combined flow is routed through circular sewers using the kinematic wave equations. Downstream flow control, backwater, flow reversal, surcharging, and pressure flow are not considered. Diversions are computed at regulators using the appropriate weir and orifice equations. The filling of overflow storage facilities is computed, but not the return of the stored wastewaters to the sewerage system.

The combined wastewater quality is routed through the sewers using pure advection, neglecting scour and deposition, dispersion, decay, reactions and interactions. Tables relating flow and concentration with treatment efficiency are used to determine wastewater quality improvement at treatment plants and overflow treatment facilities.

For realtime control of overflows during rainstorms, diversions at controllable regulators are computed using dynamic programming to maximize the utilization of available sewer, storage and treatment capacities and minimize pollutant overflows. For design studies, sizes of sewers, overflow storage facilities, treatment plants, and overflow treatment facilities are computed which will minimize costs for specified constraints on the quality of overflows and treatment plant effluents.

Considering the complexity of the model, it is comparatively easy to use. The input data are arranged in logical groups, the program, however, lacks input data diagnostics. Program improvements are necessary for the efficient implementation of the realtime control optimization. The model can be used in different modes, using various combinations of flow simulation, quality simulation, overflow control and design optimization.

Program output includes printed and plotted rainfall intensities, water levels, discharges, velocities, and concentrations. Sizes and costs of sewers, storage and treatment plants are printed for the design option. The water flow and quality simulation is programmed in Fortran IV for a DEC PDP-9 with 16K words of core memory and the realtime control and design optimization in Fortran V for a UNIVAC 1108. The two segments can be run independently or sequentially using batch mode operation. The program and draft documentation are available. An IBM 360 computer version is operated by the City of Cleveland (Pew, et al., 1972). The model has been verified on very limited data.

British Road Research Laboratory Model

The British Road Research Laboratory Model simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a converging branch sewer and open channel network (Watkins, 1962; Terstriep and Stall, 1969). The model computes surface runoff only from impervious areas and neglects the contribution of pervious areas. The model is limited to the simulation of single runoff events. Water quality, realtime control and design features are not included.

Rainfall for each catchment is computed as the weighted average of rain at several raingauges. Losses are subtracted from the rainfall according to functions of time provided as input data. The rainfall excess is routed to inlets using time-area curves computed for each catchment from Manning's equation. Dry-weather flow is provided as input data in the form of a linear function of time.

The combined flow is routed through the sewers with a storage routing technique using average travel times computed for each hydrograph from Manning's equation. Circular and rectangular pipes and trapezoidal open channels can be modeled. The model does not consider downstream flow control, backwater, flow reversal, surcharging, and pressure flow. Flow control, diversion, storage, and treatment facilities are not modeled.

The program is written in Fortran IV for an IBM 360 and is available. Program output includes printed and plotted rainfall intensities and discharges. Water levels and velocities are not computed.

The model has been tested extensively with urban hydrologic data (Stall and Terstriep, 1972). The model appears to produce satisfactory results for drainage areas of less than 13 km^2 (5 sq mi) if the impervious areas directly connected to the storm drainage system comprise more than 15 percent of the drainage area and if the frequency of the storm is less than 20 years. The model requires a minimum of input data, is easy to use, and provides fairly accurate means of computing runoff from the paved areas of urban catchments.

The Illinois State Water Survey is completing the development of a new model based on the British Road Research method which considers also the runoff contribution from pervious areas and includes a design option to size circular sewers.

Chicago Flow Simulation Program

The Chicago Flow Simulation Program is intended primarily for the simulation of large catchments consisting of both sewered and nonsewered areas (Lanyon and Jackson, 1974). It simulates the time-varying runoff in combined sewerage systems and nonurban drainage basins consisting of several catchments and a converging sewer and natural channel network. The flow routing formulation for natural channels includes provisions for flow and storage in floodplains. The model can be used for continuous simulation using hourly or smaller time steps. Water quality, realtime control and design features are not included.

Storm runoff is computed from rainfall or snowmelt from records of not more than one raingauge per subcatchment. Snowmelt is computed by a method of the U.S. Bureau of Reclamation from precipitation, daily average air temperature and daily average wind velocity. Different formulations are used for the computation of runoff for sewered and nonsewered areas.

Immediate runoff of the entire rain falling on impervious areas and a constant fraction of the rain falling on pervious areas is assumed for sewered areas. For nonsewered areas, it is assumed that the entire rain falling on impervious areas becomes runoff, and that a constant fraction of overland storage runs off during each time interval. For pervious areas of nonsewered areas, losses are subtracted from rainfall using an empirical function of rainfall, catchment shape, and soil moisture. An empirical relationship is used for continuous accounting of soil moisture considering deep percolation and evaporation. Dry-weather flow is computed from subcatchment population and hourly values of per capita contribution which are provided as input data.

The combined flow is routed in circular pipes and trapezoidal channels by a storage routing technique using Manning's equation. The trapezoidal channel formulation considers also the effects of floodplains of trapezoidal cross section. Downstream flow control, backwater, flow reversal, surcharging, and pressure flow are not considered. Storage reservoirs which store all inflow exceeding a specified maximum outflow can be simulated. No other flow control structures and diversion facilities are modeled.

The model is very easy to use, but limited in its applicability by the assumptions inherent in the precipitation-runoff computations which neglect catchment shape, slope and surface roughness for sewered areas and impervious areas of nonsewered areas, and assume a constant surface roughness for all pervious areas of nonsewered areas. Runoff and routing constants are internal to the program and may have to be changed for application of the model in different areas. Model testing on mostly nonurban areas ranging in size from 12 to 264 km^2 (4.7 to 102 sq mi) produced generally satisfactory results.

The computer program is written in Fortran IV and is available in an IBM and a CDC 6400 version. Program output includes precipitation and both tables and line printer plots of stage, discharge and storage. Flow velocities are not computed.

Chicago Hydrograph Method

The Chicago Hydrograph Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a converging sewer and open channel network (Tholin and Keifer, 1959; Keifer, et al., 1970). The model computes diameters of circular pipes for peak flows. Although it includes formulations for continuous catchment moisture accounting it can be used only for the simulation of single runoff events due to computer program limitations. Water quality and realtime control are not included.

Rainfall for each catchment is computed as the weighted average of several raingauge records or can be computed from internal design rainfall equations for different frequencies. A modified Horton's equation accounting for soil moisture changes is used to compute infiltration from rainfall for pervious areas. Losses are subtracted for both pervious and impervious areas to account for depression storage. Recovery of infiltration capacity and depression storage is simulated with internally fixed equations.

The rainfall excess is routed separately from pervious and impervious areas to the gutters by a modified storage routing method of Izzard. Gutter routing of the combined flow is also accomplished by a storage routing technique. Routing coefficients relating storage with discharge have to be provided as input data. Equations to compute them from catchment characteristics are provided in the user's manual. Dry-weather flow can be provided in the form of hydrographs as input data at any point in the channel or sewer system.

The combined flow is routed in circular pipes and trapezoidal open channels using a linear storage routing technique. The routing coefficients relating storage and discharge are computed from the peak flow of each hydrograph with Manning's equation. Downstream flow control, backwater, flow reversal, surcharging, and pressure flow are not considered. Flow control, diversion, and storage facilities are not modeled.

The model includes considerable simplifications in the routing of overland, gutter, sewer, and channel flow which would appear to limit its accuracy and are not necessary considering present state-of-the-art of flow routing techniques. The model is useful primarily for the design of circular sewer pipes. No model testing using measured urban runoff data is reported by the model developers.

The computer program is written in Fortran IV for an IBM 1130 and is available. Program output includes precipitation and discharge and sewer diameters for the design option. Water levels and flow velocities except peak velocities are not computed.

Colorado State University Urban Runoff Modeling

The Colorado State University urban runoff modeling efforts are included in this review because of special research activities which are expected to provide new and improved techniques for the control of overflows and the simulation of nonsteady flow in pipes. Two parallel efforts are being conducted. One effort is concerned with the development of concepts for the realtime control of overflows and the other with physical and numerical modeling of nonsteady flow in circular pipes.

No computer programs have been developed under the realtime control research effort which can be used directly for realtime applications. Valuable concepts evolved under this program, however, related to the future direction of urban wastewater management and the development of automated realtime wastewater control systems (Smith, et al., 1972; Grigg, et al., 1973).

Various explicit finite difference solutions of the dynamic wave equations for gradually varied nonsteady open channel flow were investigated (Yevjevich and Barnes, 1970). The diffusion scheme, Lax-Wendroff scheme, and specified interval scheme of the method of characteristics were computer programmed for a single string of circular pipes. The programs consider input hydrographs at various points, constant initial flow conditions, upstream and downstream hydraulic control, backwater, and flow reversal. Surcharging and pressure flow are not considered.

The methods were tested with data obtained from a laboratory conduit. The specified interval scheme of the method of characteristics was found to be the most accurate of all methods investigated. Implicit finite difference schemes were not investigated.

The programs of the three schemes were written in Fortran IV for a CDC 6400 and 6600 and are available. The programs compute and print water levels, velocities and discharges at any desired point along a single pipe string. The programs are not set up for pipe networks and diversion structures but merit consideration for incorporation into comprehensive urban runoff simulation models.

Corps of Engineers STORM Model

The Storage, Treatment and Overflow Model (STORM) of the Corps of Engineers Hydrologic Engineering Center is intended primarily for the evaluation of stormwater storage and treatment capacity required to reduce untreated overflows below specified values (Corps of Engineers, 1974). The model can simulate hourly stormwater runoff and quality for a single catchment for several years. Five water quality constituents are computed for different land uses: suspended and setteable solids, biochemical oxygen demand, nitrogen, and phosphorus.

Runoff is computed from hourly precipitation data of a single raingauge. The rainfall excess is defined as the difference between the available rainfall and losses to depression storage. A constant recovery rate for depression storage accounts for evapotranspiration. A weighted average of runoff coefficients for the pervious and impervious areas defines the fraction of the rainfall excess which becomes surface runoff during periods of no precipitation. Runoff from snowmelt is computed by the degree-day method.

Stormwater quality is computed from nonlinear functions considering the daily rate of dust and dirt accumulation, the percent of each pollutant contained in the dust and dirt, street sweeping practices, and days between runoff events. The BOD, N, and PO_4 runoff rates depend also on the rate of runoff of suspended and settleable solids. Land erosion is computed with the universal soil loss equation.

The model does not route the stormwater runoff and quality in a sewer or channel network. Computations of treatment, storage and overflow proceed on an hourly basis by simple runoff volume and pollutant mass balance for the entire catchment. If the hourly runoff exceeds the treatment capacity, the excess runoff is put into storage. If the storage capacity is also exceeded, the excess runoff becomes untreated overflow. If the runoff is less than the treatment capacity and water is in storage, then the excess treatment capacity is utilized to diminish the storage volume.

Plug flow is assumed for the routing of pollutants through storage. The water quality is not modified in storage. Treatment considers only the hydraulic capacity of the treatment facilities and does not compute water quality improvement.

The model depends on several empirical coefficients for both stormwater runoff and quality computations. The runoff coefficients are provided as input data, the quality coefficients are internal to the program and may have to be modified for different land uses and catchment characteristics. Verification of the water quality equations has been limited, and the verification of the complete model is still in progress. Its accuracy consequently has not been determined.

The model appears useful primarily for preliminary planning studies to estimate approximate magnitudes of untreated stormwater overflows for various combinations of storage and treatment capacities. The model does not consider costs, however, and repeated runs with different capacities are required to determine an optimal combination meeting constraints on overflows. The model is limited in its application to stormwater drainage systems since it does not consider dry-weather flow and quality.

The program is written in Fortran IV for a UNIVAC 1108 and is available. Program output includes the input precipitation data and summaries of overflow, storage, and treatment flow volumes and pollutant masses for each runoff event. Line printer plots indicating the utilization of storage are also available.

Dorsch Consult Hydrograph-Volume Method

The Dorsch Consult Hydrograph-Volume Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a sewer and open channel network consisting of loops and converging and diverging branches (Koniger, 1972; Klym, et al., 1972; Dorsch Consult, 1973). The flow routing is based on the dynamic wave equations and simulates backwater, surcharging and pressure flow but not flow reversal. The model also simulates retention basins and diversion structures. The model is limited to the simulation of single runoff events. Water quality, real-time control and design features are not included.

A diurnal pattern of dry-weather flow is computed representing base flow, constant infiltration into the sewers from groundwater, and constant sanitary flow computed from the number of residents and the per capita contribution. Storm runoff is computed separately for pervious areas and impervious areas with and without depression storage. One raingauge can be used for each subcatchment. For areas with depression storage, runoff does not begin until the smaller depressions are filled. Horton's or Holtan's equation can be used to compute infiltration on pervious areas. Evapotranspiration, snowmelt, and soil moisture accounting during periods of no rain are not considered with the exception of depression storage recovery on pervious areas. Overland and gutter flow routing is accomplished by a kinematic wave formulation.

Flow routing in sewers and open channels is accomplished with an implicit finite difference solution of the dynamic wave equations. Several cross sections are modeled. The formulation considers drop structures and retention basins and is coupled with equations for overflow diversion structures. An iterative solution technique is used which considers backwater and downstream hydraulic control but neglects flow reversal. Special formulations are included to handle surcharging and pressure flow.

This is one of the most complete models for the computation of runoff from urban catchments and the routing of flows in sewer networks. Extreme detail describing individual lots or at the most city blocks is required, however, for the accurate computation of runoff from rainfall. The implicit solution of the dynamic wave equations provides an accurate means of computing flow routing considering both upstream and downstream boundary conditions and special hydraulic structures. The numerical solution is very time consuming on the computer, however, and justified only if backwater and surcharging are significant and the effect of downstream hydraulic conditions on diversion and retention basin performance have to be considered.

This is a proprietary model of Dorsch Consult of Munich, Germany, which operates a North American branch in Toronto, Canada. The computer program is written in Fortran IV for a UNIVAC 1108, CDC 6600, and IBM 360/50. Although the model has been applied extensively in Germany, reported model testing with measured data has been limited to small catchments of less than 20 ha (50 acres). Comparisons between measured and computed runoff values have been good.

Program input and output can be either in U.S. Customery or metric units. Program output includes water levels and discharges for all system elements, plus storage volume for retention basins and cumulative inflow and outflow for both diversion structures and retention basins.

Environmental Protection Agency Storm Water Management Model

The Storm Water Management Model of the U.S. Environmental Protection Agency is one of the most comprehensive mathematical models for the simulation of storm and combined sewerage systems. It computes the combined storm and sanitary runoff from several catchments and routes the flows through a converging branch sewer network (Metcalf & Eddy, et al., 1971). It can model two types of flow diversion structures, three storage basins, and one overflow treatment plant.

Suspended and settleable solids, biochemical oxygen demand, carbonaceous oxygen demand, coliform bacteria, nitrogen, phosphorus, and oil and grease are modeled, and the performance and cost of nine unit treatment processes can be computed. A receiving water segment of the model computes the flows and water quality impact of sewerage system effluents in receiving waters. The model does not include realtime control. The model is limited to the simulation of single runoff events. The model includes an option to size sewers to eliminate surcharging.

Several raingauge records can be provided as input data but only one raingauge can be assigned to each subcatchment. Runoff from both pervious and impervious areas begins when the available depression storage is filled. Infiltration on pervious areas is computed with Horton's equation without accounting for changes in soil moisture conditions. Overland and gutter flow routing is accomplished by a kinematic wave formulation.

Stormwater quality is computed from nonlinear functions considering different land uses. The formulations consider the pollutant accumulation between storms, street sweeping practices and the rate of stormwater runoff. Dry-weather flow and quality can be provided as input data or computed from land use characteristics and account for weekday and diurnal variations.

The combined flow is routed through the sewers using the kinematic wave equations. Backwater can be approximated at up to two locations by specifying a storage element and providing appropriate geometric input data. If surcharging occurs, the model assumes that the flow in excess of full pipe flow is stored in the next upstream manhole or the sewer size can be increased to eliminate surcharging. Flow reversal is not modeled. The geometries of 12 closed conduit shapes are programmed internally and three additional aribtrary shapes can be specified by input data. Two types of diversion structures can be simulated using approximate equations for their performance. Two inline and one overflow storage facility can be modeled considering four types of outfall structures. Pumping stations with constant pumping rates can also be modeled.

The combined wastewater quality is routed through the sewers using pure advection, with scour and deposition for suspended solids and decay for biochemical oxygen demand. Either plug flow or complete mixing can be selected for the water quality routing through storage. Quality reactions and interactions are not simulated.

Nine unit treatment processes can be simulated in any series combination for a single overflow treatment facility. Functions are built into the program to compute the annual cost of overflow storage and treatment.

The receiving water portion of the model solves the one-dimensional dynamic wave equations for nonsteady gradually varied open channel flow (De Saint-Venant equations) and advective transport of pollutants for both one-dimensional and two-dimensional arbitrarily shaped grid systems. The model does not consider chemical and biological reactions and interactions between different water quality constituents.

Segments of the model have been tested on different sets of data since adequate data of rainfall, runoff and water quality were not available for a single catchment to test the complete model. Testing of the runoff and flow computations on catchments ranging in size from 5 to 2200 ha (13 to 5400 acres) showed good agreement with measured values. The accuracy of the water quality computations, particularly the formulations relating water quality with land use, has not been sufficiently established to be used with confidence for prediction purposes.

The computer program is very complex and requires a major effort for its implementation. The input data are arranged in logical groups, but improvements in the user's manual and documentation of the model's theoretical bases is needed to understand the meaning and use of some data. Program improvements, addition of new model features, and a new user's manual are in progress at the University of Florida (Heaney, et al., 1973).

The program is written in Fortran IV and versions are available for the IBM 360 and 370, UNIVAC 1108, and CDC 6400 computers. Program output includes tables and line printer plots of rainfall intensities at all raingauges, discharges and water quality for each subcatchment and for up to ten sewer system elements. Summaries of treatment performance and cost are also printed. Water levels and velocities in the sewerage system are not computed.

Hydrocomp Simulation Program

The Hydrocomp Simulation Program (Hydrocomp, 1972) is an improved version of the Stanford Watershed Model (Crawford and Linsley, 1966) which was the first comprehensive mathematical model of catchment hydrology based largely on physical concepts. Recently, a separate program was developed for the simulation of water quality in river basins (Hydrocomp, 1973) and interfaced with the hydrologic program. The program is formulated for the continuous simulation of both water flow and quality from several catchments and the routing in a converging branch sewer and open channel network. Although originally developed for nonurban areas, modifications give the program the capability to simulate both sewered and nonsewered areas. The water quality model simulates 17 water quality constituents, including their reactions and interactions in natural water bodies. The model does not include wastewater treatment, realtime control, design, and cost computations.

Precipitation for several raingauges is provided as input data to compute runoff from rainfall or snowmelt. Other meteorologic data are needed for the computation of evapotranspiration and snow accumulation and melt. Empirical equations are used to compute interception, infiltration, percolation to the groundwater, groundwater contribution to surface flow, and evapotranspiration from interception, depression storage, groundwater storage and surface waters. A Corps of Engineers method is used to compute snow accumulation and melt. The formulations require several empirical coefficients whose values have to be determined by calibration with measured data.

Both overland and channel flow routing is accomplished using kinematic wave formulations. A dispersion term in the kinematic wave equation approximates backwater effects for the channel routing. The channel routing is formulated for trapezoidal open channels with trapezoidal floodplains and for circular closed conduits. Flow control and diversion structures are not modeled by their hydraulic equations, but diversions can be specified by providing diversion hydrographs as input data. Reservoirs can be simulated in the channel network by defining their geometry and rule curves for their discharge.

Dry-weather flow and quality is provided as input data at constant time intervals. A power law is programmed as an option to compute dry-weather quality from dry-weather flow. Stormwater quality is computed with nonlinear functions of land use and stormwater runoff similar to the formulation of the EPA Storm Water Management Model. Special empirical functions are built in to compute dirt and dust accumulation between runoff events, and to account for street sweeping practices, natural decay, wind effects, and seasonal variations.

The combined wastewater quality is routed in the channels using pure advection, with a weighted mixing of concentrations between successive time steps. Water quality reactions and interactions are formulated for natural water bodies, but their application to sewers is not indicated in the model documentation. Only one-dimensional flow and quality routing is formulated. Mixing coefficients can be specified, however, for vertical water quality interchange between up to nine horizontal layers in impoundments.

Although the model was originally formulated for nonurban catchments, it was modified for application to urban catchments by adding formulations for pervious and impervious areas and circular conduits. The addition of other sewerage system phenomena and elements is needed, however, to provide the capability to simulate comprehensive networks. The hydrology formulation may be more complicated than needed for normal application to urban catchments considering the lack of available data for calibration purposes.

A major advantage of the model is the capability for continuous simulation of both water flow and quality in complex networks since it considers both catchment moisture and water quality accounting between runoff events and allows time intervals from a few minutes to bi-monthly. The hydrologic model has been tested and applied extensively in the United States and abroad in both nonurban and urban catchments. Testing of the water quality model is in progress.

The computer program is written in PL/1 language for IBM 360 and 370 computers. This is a proprietary model of Hydrocomp International, Inc. of Palo Alto, California. User's manuals are available and user's workshops are held periodically.

Program input or output can be in either metric or British units. Program output includes precipitation; soil moisture status; water stage, velocities, discharges and water quality concentrations for all channels and storage; and volume of storage.

Massachusetts Institute of Technology Urban Watershed Model

The Massachusetts Institute of Technology (MIT) Urban Watershed Model simulates the time-varying runoff of several catchments and a converging sewer and open channel network (Harley, et al., 1970). The model appears to be suitable for continuous simulation with the exception of snow accumulation and melt. Water quality and realtime control features are not included.

The model computes the sizes and costs of sewers, storage and treatment facilities which will result in the least-cost combination of alternatives for the elimination of untreated overflows and the reduction of flooding and surcharging (Kirshen, et al., 1972). The original model was developed at MIT under various projects for the U.S. Office of Water Resources Research but the model has been modified by Resource Analysis, Inc., for routine applications (Schaake, et al., 1973). Some of these modifications have not been fully documented.

Rainfall can be provided as input data for several raingauges or a single rainstorm can be moved across the catchment by defining its speed and direction of movement. Only one raingauge can be assigned to each subcatchment.

Meteorological data for a single station are used to compute evapotranspiration with Penman's equation. Four options are available for the computation of infiltration on pervious areas: Horton's equation, Holtan's equation, a U.S. Soil Conservation Service method, and a runoff coefficient method. The original MIT version computed infiltration based on the advection-diffusion equation for nonsteady one-dimensional vertical water flow in porous media.

A method based on filter theory can be used to compute the infiltration coefficients from measured rainfall and runoff (Leclerc and Schaake, 1973). Infiltration is computed from overland flow depth. The model documentation does not describe the consideration of initial losses, such as the filling of depression storage and differences in rainfall loss and runoff formulations between pervious and impervious areas.

Flow routing is accomplished with kinematic wave formulations. The equations are solved by a finite difference scheme for overland flow and flow in triangular gutters and open channels, trapezoidal open channels, and circular closed conduits. The formulations do not appear to include provisions for downstream flow control, backwater, flow reversal, surcharging, and pressure flow.

Flow control and diversion structures are not modeled. The original MIT version included an option to use a linearized solution of the dynamic wave equations if backwater conditions were considered significant and a formulation for shock waves.

The design option of the model computes the sizes and costs of circular sewers, the volume of overflow storage basins, and the hydraulic capacity of treatment plants needed to reduce undesirable flooding and surcharging and to eliminate untreated overflows. Linear programming is used to determine the least-cost combination of these facilities for a design storm event. The design does not consider water quality.

Model documentation is scattered through several project reports and a draft user's manual was not released. It is therefore difficult to evaluate the model since its present status is not fully documented.

The model is based on physical principles and includes a minimum of approximations. Of particular interest are its capability to compute infiltration coefficients from measured rainfall and runoff data and the design option which considers the dynamic response of the catchment and sewerage system runoff to rainfall. Testing on the 9 ha (23 acres) Gray Haven catchment near Baltimore, Maryland, produced good agreement between measured and computed runoff values.

The computer program was written for an IBM 360/67 computer. The design option requires the IBM linear programming package (MPSX 1971). The user may need to contract with Resource Analysis, Inc. of Cambridge, Massachusetts, for routine application of the complete and revised program package.

Computer output includes tables of rainfall intensities, and overland, catchment and channel depth and discharge. Flow velocities are not printed. The output of the design option appears to include the volume and duration of flooding for each sewer and the costs and sizes of sewers, overflow storage facilities and treatment plants.

Minneapolis-Saint Paul Urban Runoff Model

The Minneapolis-Saint Paul Urban Runoff Model was developed for realtime forecasting of flows in the major trunk and interceptor sewers of the Minneapolis-Saint Paul combined sewer system. Its purpose was to compute regulator settings which would reduce untreated overflows during rainstorms to the Mississippi River (Minneapolis-Saint Paul Sanitary District, 1971). The model computes the runoff of several large catchments, diverts the flows at controllable regulating structures, and routes them through a converging branch sewerage network to the treatment plant. The model is specifically designed for realtime control operation on a small computer and cannot be used easily for the assessment of existing or the design of new sewerage systems since it uses a highly simplified flow routing procedure whose coefficients have to be calibrated with measured data or derived from more sophisticated models.

Although the model was designed for continuous simulation by incorporating provisions for the accounting of catchment moisture between rainstorms, the necessary formulations were never added and the program can be used only for the simulation of single runoff events. The realtime control operation of the model is not based on mathematical optimization techniques but requires repeated trial and error runs with estimates of regulator settings. The model does not include water quality, design and cost computations.

Rainfall for each catchment is computed as the weighted average of rain at several raingauges. Losses are subtracted from rainfall for both pervious and impervious areas. An exponential decay function computes losses on impervious areas, and a modified Holtan's equation computes infiltration on pervious areas, both formulations accounting for changes in catchment moisture conditions. Unit hydrographs derived from catchment characteristics by a method of the U.S. Soil Conservation Service are used to compute runoff from the rainfall excess. Dry-weather flow considering diurnal (hourly) variations is provided as input data.

The combined wastewater flow is routed through the sewers with the progressive average lag method which requires two empirical routing coefficients for each pipe. The formulation does not consider downstream flow control, backwater, flow reversal, surcharging, and pressure flow. The coefficients have to be derived from measurements of flow. Since these were not available, a solution of the dynamic wave equations for a single pipe was used to compute them for each pipe of the Minneapolis-Saint Paul interceptor sewers.

Diversions are computed at six types of regulators by using the appropriate combinations of weir and orifice or gate discharge equations. The equations are formulated for controllable regulators, and gate settings and inflatable weir heights can be read in as functions of time. Other flow control facilities, such as pumping stations and storage facilities, are not modeled.

The computer program was written in Fortran IV to be run on a DEC PDP-9 computer with 16K words of core memory at half hour intervals for realtime predictions. Consequently, several simplifications were necessary in the mathematical formulations to stay within these constraints. These included the use of the unit hydrograph method for the rainfall-runoff computations, the lumping of small catchments into larger catchments, and the use of the progressive average lag method for the flow routing. These methods, particularly the routing method, limit the model's accuracy and its applicability to other areas without extensive calibration.

Of particular interest, however, is the interfacing of the model with a realtime data acquisition system for rainfall, water level, and regulator status data, and the remote control of the regulator gates and weirs. The model is not used for realtime control anymore, however, since the required trial and error procedure with estimates of regulator settings is too time-consuming, and operator experience with the system appears to be more efficient for the control of the regulators.

Seattle Computer Augmented Treatment and Disposal System

The Computer Augmented Treatment and Disposal System (CATAD) of the Municipality of Metropolitan Seattle (Metro) is an operating system for realtime control of untreated overflows from the main trunk and interceptor sewer regulators of the metropolitan Seattle, Washington, combined sewerage system (Municipality of Metropolitan Seattle, 1971; Gibbs, et al., 1972; Mallory and Leiser, 1973). The system does not include a comprehensive mathematical model for the simulation of runoff from several catchments and the routing of flows in a sewerage network. A separate model with these functions is being developed for interfacing with the realtime control scheme. Present system operation is based on independent computer control of regulators and pumping stations using realtime data acquisition during rainstorms and rule curve operation. Water quality is not considered by the scheme.

The system provides automatic computer control of 16 regulators and two pumping stations. The remote control is determined using realtime data acquisition of rainfall from six raingauges and water levels upstream and downstream of the controllable regulators and pumping stations. Rule curves are used to determine the operation of the regulators and pumping stations which will maximize the available sewer storage capacity and minimize untreated overflows.

The rule curves represent sewer storage capacity as functions of time for the sewer upstream of each controlled regulator and pumping station. Each curve was developed from typical rainstorm patterns and represents the desirable filling schedule for a sewer based on its storage capacity. The rule curves are adjusted during realtime operation to reflect actual rainfall patterns. Hydraulic equations have been programmed for each regulator to determine the gate settings which will produce the sewer filling schedule and flow diversion computed by the rule curves. At present, each regulator is controlled independently, and the effect of flow routing on overall system performance is not considered. Manual override is possible for alarm conditions.

The computer program is written in Fortran for a Xerox Sigma 2 computer with 49K words of core memory. The realtime data acquisition and remote control operation (foreground operation) requires 26K while the remaining 13K are available for other purposes. The control scheme uses simulation at half hour intervals to compute the required regulator and pumping station operation for the following half hour.

Evaluation of the control scheme has shown significant reductions in untreated overflows compared to uncontrolled conditions. The scheme's effectiveness is impaired, however, since each regulator is considered independently rather than as part of the whole sewerage system, and since it neglects the effect of the quality of the overflowed wastewaters on the receiving water quality.

SOGREAH Looped Sewer Model

The Looped Sewer Model of the French consulting firm Societe Grenobloise d'Etudes et d'Application Hydrauliques (SOGREAH) simulates the time-varying runoff of combined sewerage systems consisting of several catchments and a sewer and open channel network including loops and converging and diverging branches (SOGREAH, 1973 - 4 reports). The model includes formulations for most hydraulic phenomena encountered in closed conduit and open channel networks. The flow routing solves the dynamic wave equations coupled with equations for special sewer system facilities, such as diversion structures, pumping stations, inverted siphons, and retention basins. The solution considers both upstream and downstream boundary conditions, backwater, flow reversal, surcharging, and pressure flow. The model has been expanded recently to include the advective transport of conservative pollutants. The model does not include realtime control, design and cost computations.

The formulation for the catchment runoff from rainfall is limited to the simulation of single runoff events, the flow routing portion of the model, however, can be used for continuous simulation. This is a proprietary model and not all details of the model formulations were available.

The model does not include a provision to read in rainfall data but computes design rainfall excess hyetographs of specified frequency for each catchment using formulas developed by Caquot. The formulas consider catchment area, slope and percent imperviousness and require ten empirical coefficients which is a serious limitation for application of the model to other than French hydrologic conditions. The design hyetographs are routed to catchment inlets with the Muskingum flood routing method which requires the estimation of two additional coefficients. One of the coefficients is computed by a formula suggested by Schaake and Geyer which considers catchment length, slope, and percent imperviousness and uses four additional empirical coefficients. Dry-weather flow is provided as input data in the form of hydrographs at constant time intervals.

Flow routing in sewers and open channels is accomplished with the dynamic wave equations. An implicit finite difference solution is coupled with hydraulic equations for special sewerage system facilities, such as gates, orifices, weirs, inverted siphons, pumping stations, and retention basins. The solution considers both upstream and downstream flow conditions at all facilities and in the sewers, backwater, flow reversal, surcharging, pressure flow, and flooding at inlets or manholes. The geometry of circular and egg-shaped pipes and of trapezoidal open channels is programmed internally and any arbitrary shape can be defined as input data or by the addition of appropriate subroutines. The flow routing is set up for continuous simulation.

The model would be needed primarily for the simulation of systems where the interactions of all modeled hydraulic phenomena are significant and where loops and diverging branches need to be modeled in addition to converging branches. A major model weakness is its formulation of the rainfall-runoff process. Substitution by a different approach based on more physical principles would be advisable. The flow routing computations, due to their completeness, require much computer time, some of which can be saved by specifying different lengths of time steps depending on the expected rate of change of flow.

The model routing scheme is based on a river basin model developed by SOGREAH earlier. The firm did not verify the sewer model with urban hydrologic data since the river basin model verification produced satisfactory results.

The computer program consists of five main programs written in Fortran IV for an IBM 360/65. This is a proprietary model of SOGREAH of Grenoble, France. The firm does not have a North American representative at present.

Program output includes tables of water depths (or pressures), velocities and discharges, and plots of water depths and discharges for each computational point. Program input and output is in metric units.

University of Cincinnati Urban Runoff Model

The University of Cincinnati Urban Runoff Model simulates the time-varying runoff of storm sewerage systems consisting of several catchments and a converging branch sewer and open channel network (University of Cincinnati, 1970; Papadakis and Preul, 1972). The model does not include provisions for dry-weather flow, water quality, realtime control, and design. It is limited to the simulation of single runoff events.

The University of Cincinnati Urban Runoff Model accepts input of only one hyetograph for the entire drainage basin. Although this could be the weighted average of several raingauge records, it neglects the areal non-uniformity of rainfall. Losses are subtracted from rainfall separately for pervious and impervious areas to account for depression storage. Infiltration on pervious areas is subtracted from rainfall using Horton's equation, with its time origin offset to balance cumulative rainfall with potential infiltration.

Overland flow is computed with a storage routing technique and Manning's equation. Gutter flow routing uses a steady-state approach assuming that the gutter outflow equals the inflow during the same time interval. Flow routing in circular sewers is accomplished by a simple translation of the upstream hydrograph by an average travel time computed with Manning's equation.

The model includes several simplifications which appear to reduce the model's accuracy and applicability. This includes the assumption of areal uniformity of rainfall, the neglect of dry-weather or base flows, the computation of infiltration from rainfall rather than overland flow depth, the neglect of catchment moisture conditions in the infiltration computations, the steady-state formulation for gutter flow routing, and the neglect of dynamic effects in the sewer flow routing.

Although some encouraging results are reported by the model developers (Papadakis and Preul, 1973) on a 5.2 ha (12.9 acre) and 964 ha (2380 acre) catchment, the tests were restricted to fairly simple rainfall events and required considerable calibration of the infiltration rate coefficients. Testing by others on a 70 ha (173 acre) and 500 ha (1240 acre) catchment in Australia using more complex storm patterns showed considerable differences between measured and computed runoff values (Heeps and Mein, 1974).

The computer program is written in Fortran IV for an IBM 360 and available from the University of Cincinnati. A draft user's manual is also available. Program output includes primarily discharges at selected points. Water depths and velocities are not printed.

University of Illinois Storm Sewer System Simulation Model

The University of Illinois Storm Sewer System Simulation Model computes nonsteady flows in a converging sewerage network based on a solution of the dynamic wave equations (Sevuk, 1973; Sevuk, et al., 1973). The solution considers upstream and downstream flow controls, backwater, and flow reversal in circular sewers and in-line storage at sewer junctions. The model does not compute dry-weather flows from land use or runoff from precipitation but requires inflow hydrographs to the sewers as input data. Separate general hydrologic catchment models have been developed, however (Chow and Kulandaiswamy, 1972; Chow and Ben-Zvi, 1973), and research for the development of an urban hydrologic model is in progress for potential interfacing with the flow routing model. The routing model includes a feature for the sizing of circular pipes to accommodate peak flows. Costs are not considered. The model does not include water quality and realtime control features. A separate model is being developed for sewer design using dynamic programming with cost and risk considerations.

The flow routing is based on a first-order explicit finite difference solution of the characteristic equations of the dynamic wave equations. Surcharging and pressure flow are not computed. Pumping rates, gate or weir control and diversions can be specified at the sewerage system outlet, but not at other locations. The solution technique employs an overlapping segment scheme which allows modeling large networks in segments. The scheme requires two iterations for its solution, but some of the resulting sacrifice in computer time can be saved by specifying different time steps in different segments.

The design option of the routing model is based on hydraulic concepts only and does not consider costs. An iterative approach is used starting with the first upstream pipe of each branch, then sizing pipes in each branch using a kinematic wave routing, and finally improving the sizes by routing the flows with the full dynamic equations.

The model represents a comprehensive formulation of open channel flow routing in circular sewers, but neglects special features such as pressure flow, loops and diverging branches, and diversion structures to be generally applicable. Long computer times have to be expected for the explicit solution of the dynamic wave equations. Although the model has continuous simulation capability, the cost of running it would normally restrict its application to individual design events. The increased accuracy in the flow routing may therefore be offset by uncertainties with respect to defining satisfactory design inflow events.

The model has not been verified with real sewerage network data or applied to real systems. Testing of the routing scheme with experimental pipe data indicates, however, that high accuracy can be expected.

The computer program is written in PL/1 and assembler language for an IBM 360/75 system. The program and a user's manual are available from the University of Illinois. The program is adaptable to IBM 360 and 370 systems but would require major reprogramming for implementation on other makes of computers. Program output includes line printer plots of inflow hydrographs, tables and Calcomp plots of depths and discharges, and tables of velocities at selected points of the network, and sewer diameters for the design option.

University of Massachusetts Combined Sewer Control Simulation Model

The University of Massachusetts Combined Sewer Control Simulation Model simulates the time-varying runoff of several catchments and a single string of circular sewers (Ray, 1972). The model computes runoff from impervious areas only using hourly rainfall data. This restricts the model to the simulation of runoff from large catchments with negligible surface runoff contributions from pervious areas. A separate model is available to compute synthetic hourly rainfall from recorded rainfall using a Markov chain model.

The flow routing is accomplished with an implicit solution of the dynamic wave equations which considers upstream and downstream flow control, backwater, and flow reversal. Special sewerage system facilities, such as diversion and flow control structures and storage facilities are not modeled. The model is formulated for the continuous simulation of runoff for periods of up to one month, but neglects catchment moisture balance between runoff events and snow accumulation and melt. Water quality, realtime control, and design features are not included.

Dry-weather flow is defined as a constant value for each subcatchment. Runoff is computed from impervious areas only, and it is assumed that all rainfall during an hour becomes runoff during the same hour after an initial loss is satisfied. Synthetic hourly rainfall can be computed from recorded rainfall data with a separate program. A first order Markov chain computes hourly rainfall values during wet periods, and a sixth order Markov chain computes the duration of wet and dry periods.

The combined wastewater flow is routed through a single string of circular pipes with multiple inlets for the runoff from the modeled subcatchments. The flow routing is accomplished by an implicit finite difference solution of the dynamic wave equations which consider upstream and downstream flow control, backwater, and flow reversal. Special boundary conditions, such as the simulation of flow control and diversion structures, are not incorporated.

The model has not been tested with real catchment data. Testing with hypothetical data indicated that the combination of hourly runoff computation from rainfall with the implicit flow routing scheme may be a practical combination for the simulation of large catchments and trunk and interceptor sewers where short duration peak discharges may be negligible. Considerable additions would be needed to the program to make it generally applicable, such as the computation of runoff from pervious areas, the consideration of a time lag between rainfall and runoff, the simulation of evapotranspiration and snow accumulation and melt, and the expansion of the routing scheme to sewer networks and special sewerage system facilities.

The computer programs are written for a CDC 3600 computer and are available. A user's manual has not been written. Computer output consists of rainfall intensities for each catchment, and flow depths and discharges at selected points of the pipe string.

Water Resources Engineers Storm Water Management Model

The Storm Water Management Model of Water Resources Engineers, Inc. (WRE) is a modified version of the Storm Water Management Model of the Environmental Protection Agency. The model simulates the time-varying combined storm and dry-weather runoff and wastewater quality of several catchments and a sewer and open channel network including loops and converging and diverging branches (Shubinski and Roesner, 1973).

The flow routing is based on the dynamic wave equations and considers both upstream and downstream control, backwater, and flow reversal. A special formulation computes surcharging and pressure flow independently for each junction. The solution is coupled with hydraulic equations for flow control and diversion structures and pumping stations.

Both dry-weather and stormwater quality are computed for 23 constituents: suspended and setteable solids, biochemical oxygen demand, nitrogen, phosphorus, oil and grease, and 17 arbitrary conservative constituents. The pollutants are routed through the sewerage system, but treatment processes are not modeled.

The model does not include realtime control, design, and cost computations. It is limited to the simulation of single runoff events. This is a proprietary model and not all details of the model formulation and computer program were available. A separate model for receiving water flow and quality is available but it is not interfaced with the WRE Storm Water Management Model (Chen and Orlob, 1972).

Records of several raingauges can be read in and up to three raingauges can be assigned to a single subcatchment. Depression storage, infiltration, and overland flow are computed separately for pervious and impervious areas.

Runoff begins when all depression storage is filled. A constant loss rate can be specified for impervious areas. Infiltration on pervious areas is computed with Horton's equation without accounting for changes in soil moisture. Overland flow is computed by a kinematic wave formulation. Special provisions allow the input of data for average urban blocks, the number of blocks per subcatchment, and the length of each subcatchment to allow larger spatial discretizations of the modeled catchments without significant loss in the accuracy of the runoff computations.

Stormwater quality is computed from nonlinear functions for different land uses. The formulations consider the pollutant accumulation between storms, street cleaning practices and the rate of stormwater runoff. Average values of dry-weather flow and quality is provided as input data and varied by internal functions to account for time and land use differences.

The combined flow is routed through the sewers using the dynamic wave equations. A special explicit finite difference formulation solves the continuity equation at nodes (sewer junctions and connections) and the momentum equation along links (sewer and channel reaches). The formulation computes backwater and flow reversal and is coupled with hydraulic equations for weirs, gates, orifices, and pumping stations. The geometries of five closed conduit shapes and a trapezoidal open channel are modeled. Surcharging and pressure flow conditions in the sewer network are considered independently at each junction.

The combined wastewater quality is routed through the sewers using pure advection. Dispersion, sedimentation and scour, decay, reactions and interactions are not simulated.

The model represents a very comprehensive formulation of sewerage system flow phenomena coupled with water quality computations. The explicit solution of the dynamic wave equations requires very small time steps and is time-consuming on the computer. The model would be needed where the simulation of most modeled hydraulic phenomena is important and water quality routing is desired. The computation of water quality from land use characteristics has not been sufficiently verified.

Model testing showed good agreement between measured and computed runoff values for a 19 ha (47 acre) and 1540 ha (3800 acre) catchment. Water quality predictions were of the right order of magnitude.

The computer program consists of three main programs and is written in UNIVAC 1108, IBM 360/65, and CDC 6600 versions. This is a proprietary model of Water Resources Engineers of Walnut Creek, California. Program output includes tables and Calcomp plots of depths, velocities, discharges and water quality (concentrations and mass rates) for the subcatchments, the sewer system outlet, and selected internal subcatchment and sewerage system points. Calcomp plots of depths and discharges can also be obtained.

Wilsey and Ham Urban Watershed System

The Wilsey and Ham Urban Watershed System computes the time-varying stormwater runoff of several catchments and a converging branch sewerage network (Amorocho, 1972). A design option sizes circular pipes for peak flows. The model does not include provisions for dry-weather flow, water quality, and realtime control. The model is limited to the simulation of single runoff events. This is a proprietary model and not all details of the model formulations are available.

Stormwater runoff is computed separately for pervious and impervious areas from rainfall intensities of a single raingauge for the entire modeled catchment. Losses are subtracted from rainfall, but details of the loss computations are not available. Kinematic wave formulations are used for the routing of overland flow to the gutters, and for flow routing in standard shapes of gutters and in circular sewers. A special feature routes catchbasin inflow exceeding the free flow capacity of the sewer in the gutter to the next downstream catchbasin. Downstream flow control, backwater, flow reversal, surcharging, and pressure flow are not modeled. The model does not simulate flow control and diversion structures.

A special design option sizes circular sewer pipes to accommodate the upstream inflow under free flow conditions. Costs are not considered in the design.

The model includes special provisions which considerably reduce the input data requirements and computer running time. Data need to be defined only for typical urban subcatchment elements, rather than all elements, and the appropriate hydrologic computations are performed only for these typical elements. The flow routing, however, considers the actual location of the elements.

Although sufficient details are not available on the mathematical formulations of the model, it appears to be an efficient model for the evaluation and design of small storm sewerage systems for which the limitations of the flow routing scheme are acceptable. The program has been applied extensively by Wilsey and Ham, but testing with real catchment data to evaluate model accuracy has not been reported.

Details describing the computer program, including estimates of running times, are not available. The program is written in two versions, to run on a CDC 6600 computer and on Tymshare's XDS 940 system. This is a proprietary model of Wilsey and Ham, Inc. of Foster City, California. Computer output includes discharges at selected points of the storm sewerage network and pipe diameters for the design option.

MODEL TESTING

Most reviewed models were tested and verified by the model developers and subsequent model users on very limited data as a result of the unavailability of sufficiently complete and reliable urban catchment, precipitation, runoff, and water quality data. Quite commonly, only portions of a model were tested on measured data and not the complete model. Sometimes different portions of the same model were tested with data from different catchments since comprehensive data were not available from a single catchment, especially when both flow and quality data were required. Only in rare instances were two or more models tested on the same data.

Numerical testing of the seven more comprehensive models was therefore conducted in this study using both hypothetical and real catchment data to compare model performance. The purpose of the testing was to show that different models with different mathematical formulations for the same phenomena will produce differing results given the same input data. The testing consequently provides useful information to the model users by indicating ranges of applicability of each model. Testing with the hypothetical data shows the sensitivity of each model to model parameter variations, while testing with real catchment data also provides information on model accuracy.

The reviewed models require widely varying detail with respect to the spatial discretization of data describing the catchment and sewerage system. Some models require extremely detailed information such as sizes and slopes of individual roofs, driveways, lawns, gutters, etc., while others can lump areas of several hundred acres into single subcatchments. Great differences exist also between the models with respect to the required time discretization for precipitation, runoff and water quality data. Some models require time steps of less than one minute to satisfy numerical stability conditions, while others can be run with hourly, daily, and up to semi-monthly data.

The model testing with real catchment data required the collection of available data on urban catchment and sewer system characteristics, precipitation, runoff, and wastewater quality. A large number of data sources were investigated and data for several U.S. cities were collected. Selected data were digitized in the computer formats required by the models being tested. It became rather difficult, however, to find real catchment data meeting the requirements of all models. Additional difficulties which were encountered included missing information on physical characteristics of the subcatchments and sewerage systems, uncertainties with respect to watershed infiltration characteristics and moisture conditions, insufficient documentation with respect to the instrumentation and data analysis techniques used during the measurement periods, and uncertainties with respect to the accuracy of the measured rainfall, runoff, and water quality data.

Great reluctance was expressed by model developers, therefore, to run real catchment data whose accuracy was uncertain and over whose collection they had no control. The use of hypothetical data removed these uncertainties.

Consequently, the model testing concentrated on hypothetical data. All seven models were tested using the same set of hypothetical data, but only the Battelle, Chicago Flow Simulation, EPA Storm Water Management and Dorsch models were also tested on the same set of real catchment data.

The numerical testing concentrated only on the most common features of the selected models, that is the rainfall-runoff simulations and the flow and quality routing. Special features such as snowmelt, the simulation of treatment processes, realtime control and design aspects were not tested. The hypothetical data testing was applied separately to the rainfall-runoff formulations and the routing procedures to test the response of the formulations of each phenomenon to physical parameter variations. Testing with the real catchment data served the objective of testing the interactions of the runoff computations for several catchments and the routing of flows and quality through a sewer network.

Hypothetical Data Tests

The hypothetical data runs were not intended to portray accuracy, but to determine the sensitivity of the models to parameter variations. The data were also selected to determine limits of applicability of the models, such as the ability to handle surcharging and backwater conditions. In addition, the hypothetical data runs provided information with regard to the cost of running each model.

Two independent sets of hypothetical data were selected. One set tests the rainfall-runoff relationship for various catchment conditions. The second set tests the sewer flow routing (and water quality routing, if applicable) for various combinations of pipe parameters.

Examples of hypothetical data runs are presented only for the Battelle Urban Wastewater Management Model (BUWMM), EPA Storm Water Management Model (SWMM) and Chicago Flow Simulation Program (FSP) since results from the proprietary model runs were not available in time for incorporation in this report.

Hypothetical Catchment Tests

Hypothetical catchment data tests were performed for two different catchment sizes, two orientations of each catchment with respect to the direction of surface flow, ten combinations of physical catchment characteristics, two different initial catchment moisture conditions, and four different rainstorms, representing a total of 320 data combinations. Five-minute time steps were selected for the simulation runs. The effect of different time discretizations on computed runoff was not tested.

Five examples are described here to illustrate model differences. They are for a 2-hour triangular storm with a peak rainfall intensity of 50.8 mm/hr (2.0 in/hr) and a 4.65 ha (11.5 acre) rectangular catchment. The catchment width in the direction of surface flow is 152 m (500 ft) and the catchment length perpendicular to the direction of surface flow is 305 m (1000 ft). A Manning's roughness coefficient of 0.25 was assumed for the pervious area and 0.025 for the impervious area of the catchment. Initially dry catchment moisture conditions were assumed. For pervious areas, the maximum infiltration rate is 50.8 mm/hr (2.0 in/hr), the minimum infiltration rate 12.7 mm/hr (0.5 in/hr), and the infiltration decay rate 0.001 sec^{-1} as defined by Horton's infiltration equation. The retention storage capacity for pervious areas is 5.1 mm (0.20 in.) and for impervious areas 1.3 mm (0.05 in.). The selected examples are for different combinations of catchment slope (0.1 and 10 percent) and impervious areas (0, 50 and 100 percent).

Considerable differences exist between the models in the computed runoff for the catchments with 0 percent imperviousness as a result of differences in the mathematical formulations for infiltration (Figures 1 and 4). The differences are particularly great for the 0.1 percent slope (Figure 1). For both slopes, runoff computed by the BUWMM is considerably higher than for the SWMM. The reason appears to be that Horton's equation as used in the SWMM neglects catchment moisture conditions in the computation of potential infiltration. When the ground becomes rapidly saturated, Horton's equation overestimates infiltration and underestimates surface runoff. For steeper slopes where runoff occurs more rapidly, or low intensity rain, this effect is less pronounced since less water is available for infiltration.

Holtan's equation as used by the BUWMM accounts for changes in soil moisture, but on the other hand introduces a different approximation by computing potential infiltration from rainfall rather than the depth of water on the catchment. It therefore underestimates infiltration for flat slopes where runoff is sufficiently slow and rainfall occurring during one time step contributes to infiltration not only during the same but also succeeding time steps.

The FSP assumes that a constant fraction of the rain falling on the pervious areas of sewered catchments runs off during the same time step and neglects catchment slope, shape, and roughness. For the FSP simulations presented here, nonsewered catchment runoff was specified, however, which computes losses from rainfall with an empirical equation accounting for soil moisture changes and evaporation and considers catchment shape and slope. For the flat slope (Figure 1) the FSP computes an extremely low and slow response, and the runoff hydrograph does not recede during the 6-hour simulation period.

Differences in the computed results are introduced also by the overland flow routing procedures. The BUWMM uses a unit hydrograph approach, the SWMM a kinematic wave formulation, and the FSP a linear storage routing which considers catchment shape and slope but neglects surface roughness.

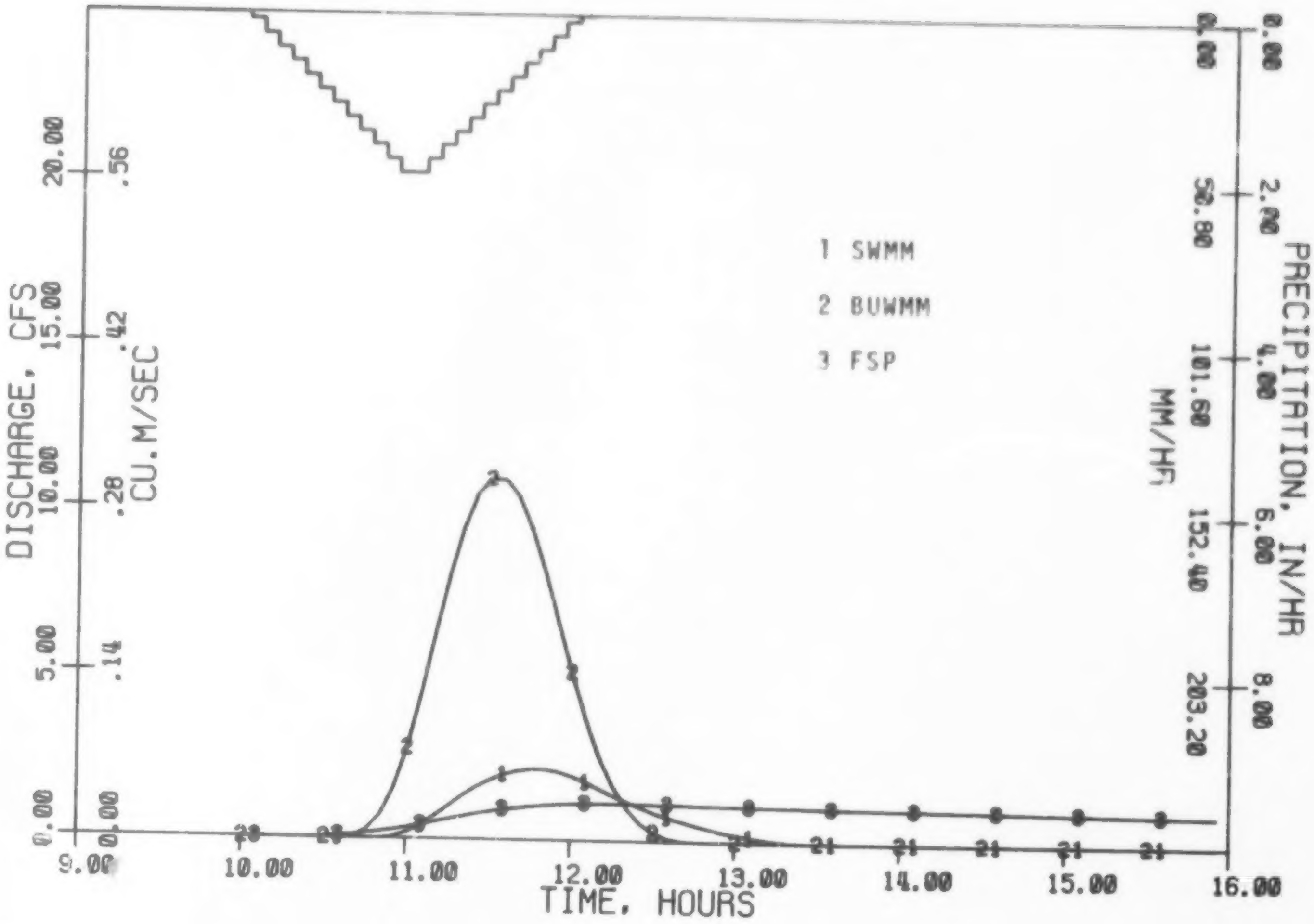

Figure 1. Hypothetical catchment simulation - 0.1% slope and 0% impervious

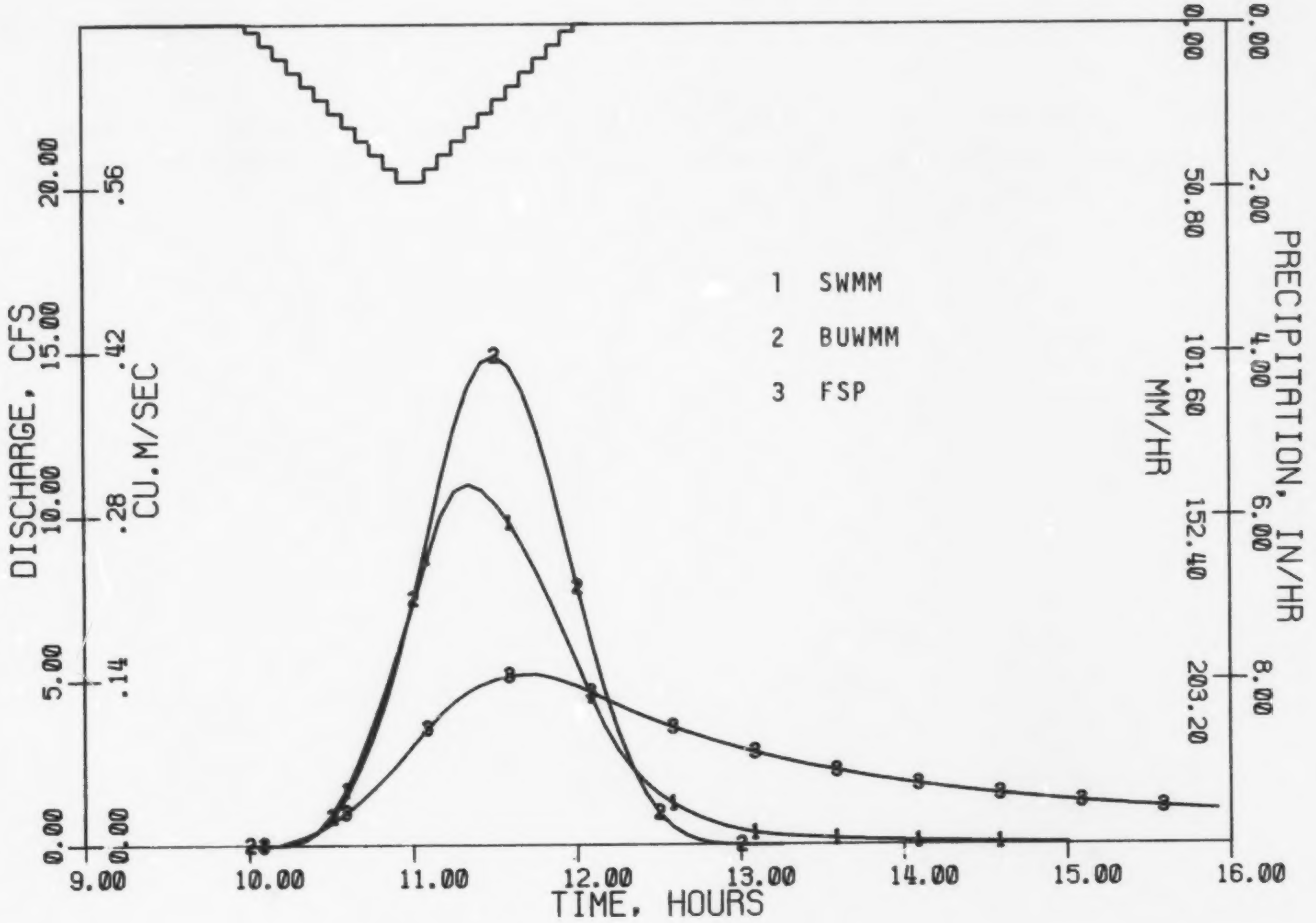

Figure 2. Hypothetical catchment simulation - 0.1% slope and 50% impervious

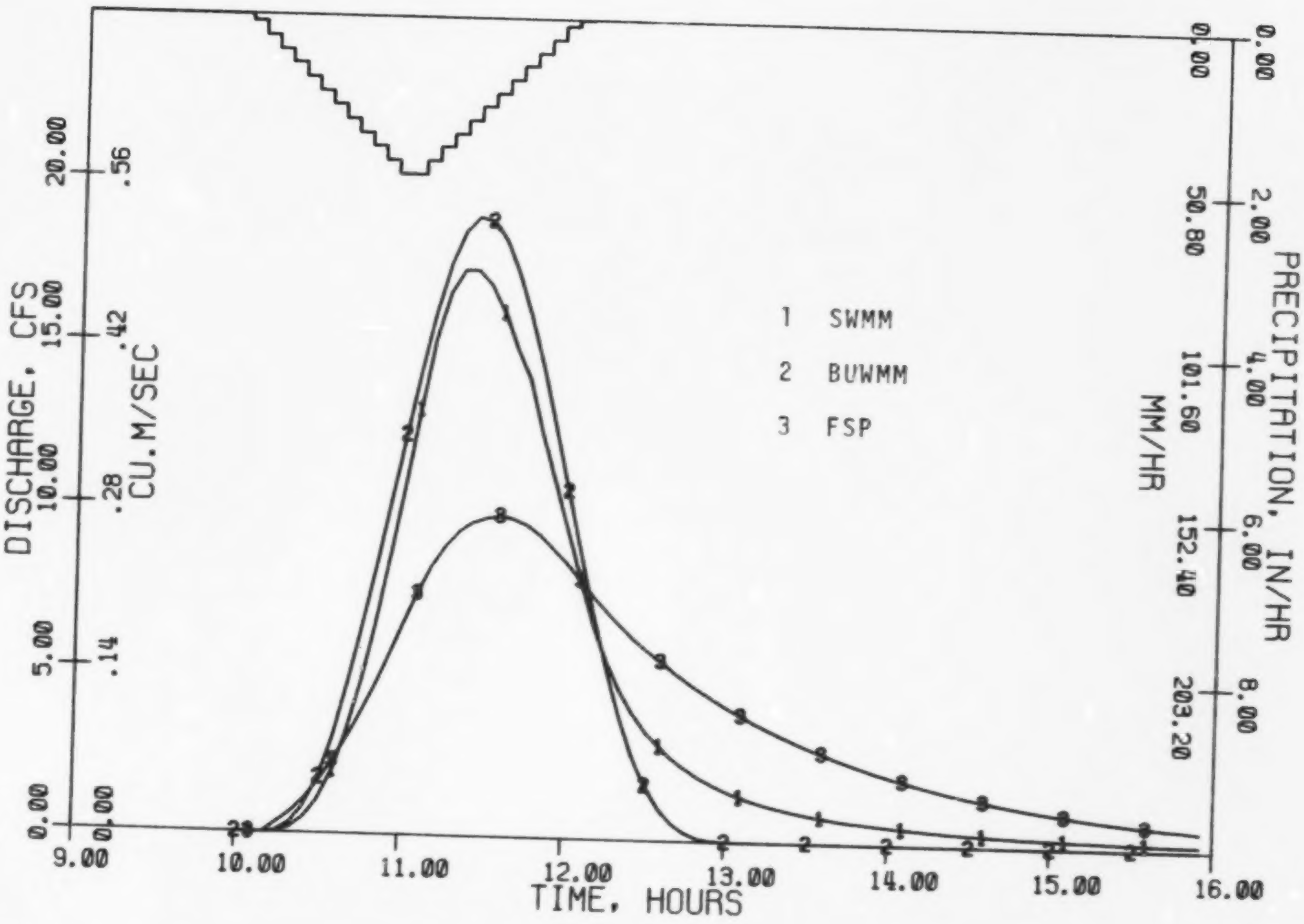

Figure 3. Hypothetical catchment simulation - 0.1% slope and 100% impervious

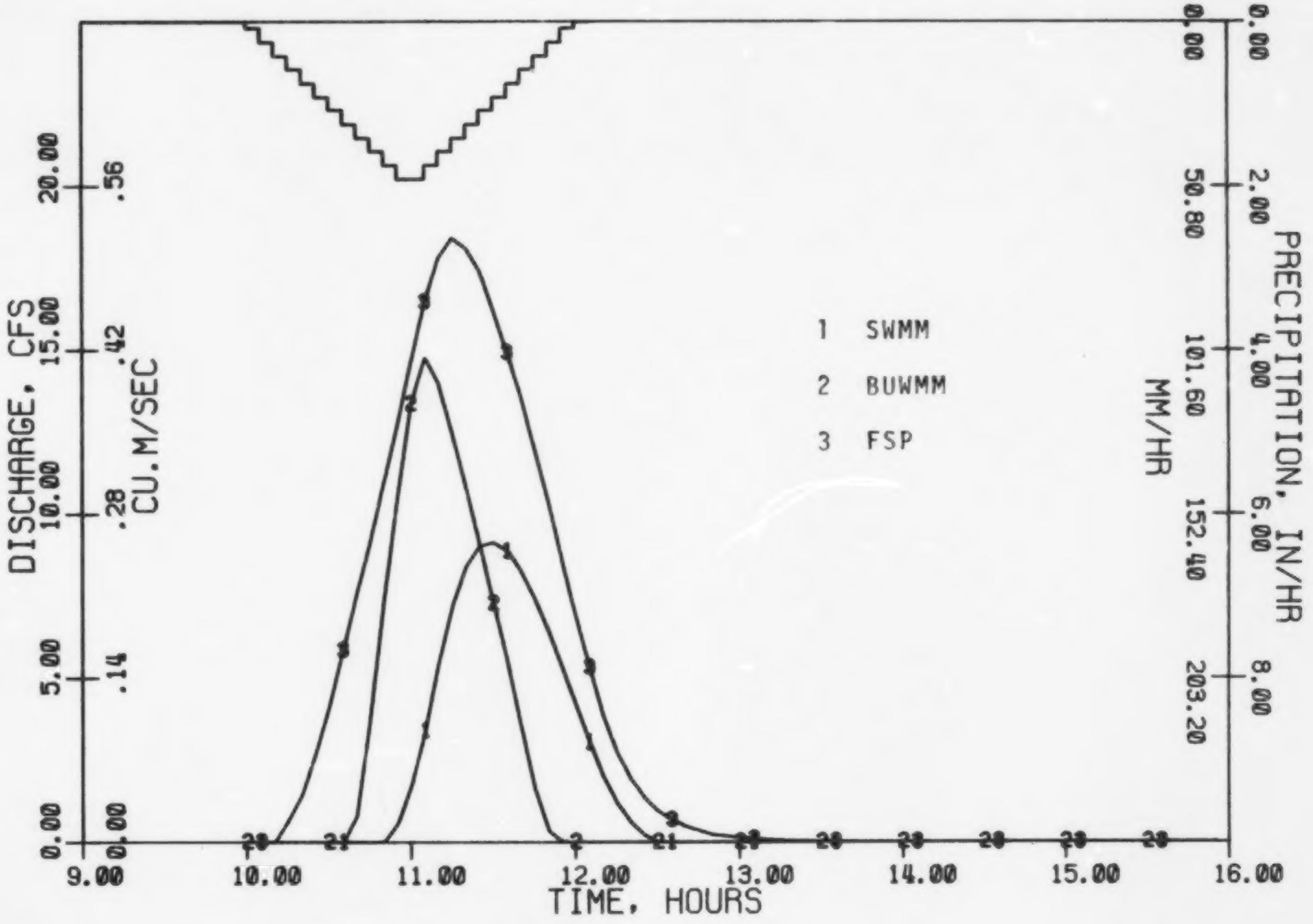

Figure 4. Hypothetical catchment simulation - 10% slope and 0% impervious

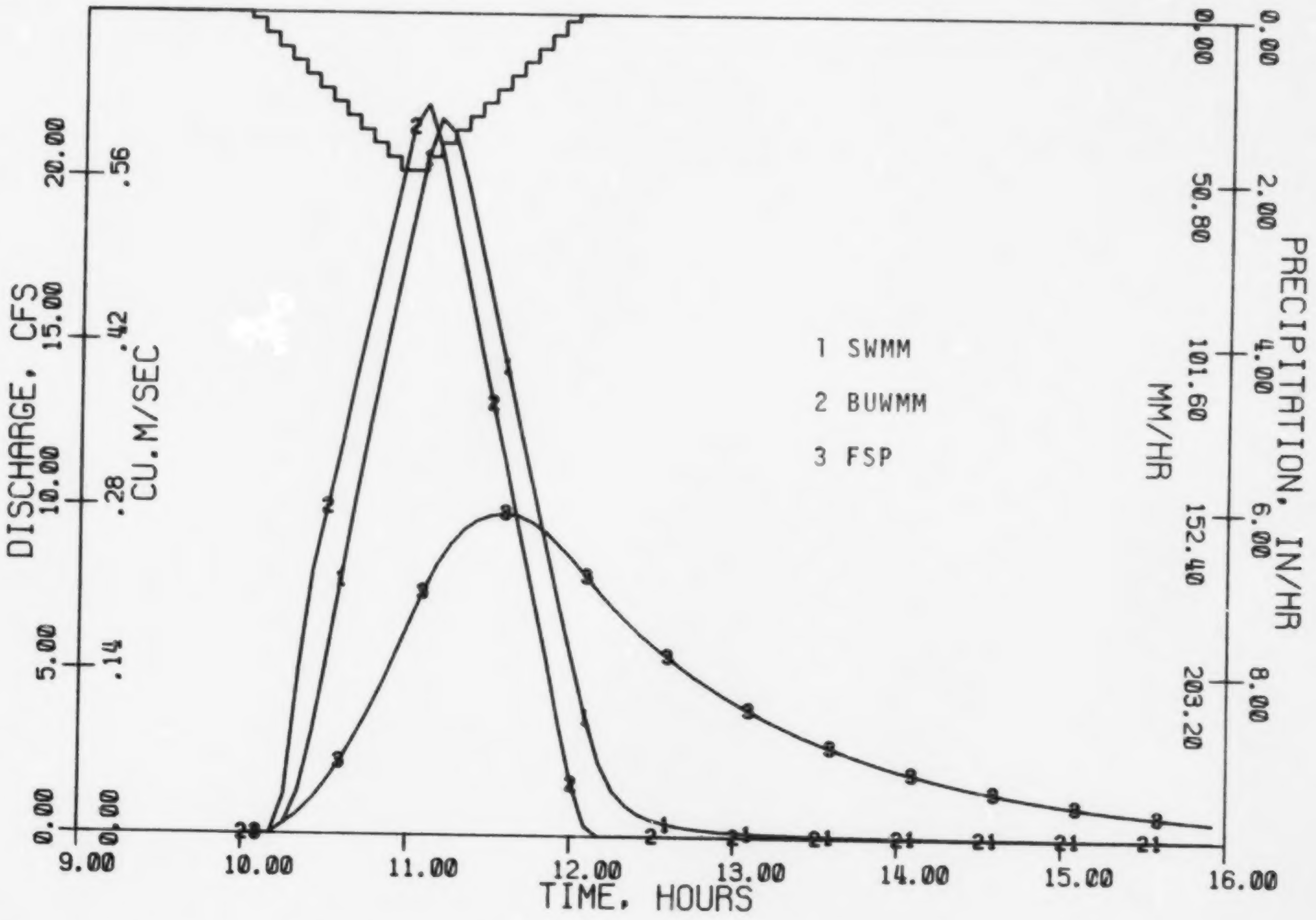

Figure 5. Hypothetical catchment simulation - 10% slope and 100% impervious

The effect of these differences can be seen by comparing the results of the runoff computations for the 100 percent impervious areas, since for these runs rainfall losses were considered negligible and the computed differences are primarily the result of differences in the overland flow formulations. As Figures 3 and 5 show, the results of the BUWMM model and SWMM computations are quite comparable for the steep slope (Figure 5) and show only small differences for the flat slope (Figure 3). The FSP assumes that a constant fraction of the overland flow storage on nonsewered areas runs off during each time step. The resulting runoff hydrograph is much flatter and longer than the BUWMM and SWMM hydrographs. Since the FSP formulation neglects catchment shape, slope, and surface roughness for impervious areas, the computed runoff hydrographs for the flat and steep slopes are the same.

The combined effects of the formulations for infiltration and overland flow for pervious and impervious areas is illustrated in Figure 2 for the 0.1 percent slope and a 50 percent imperviousness. The BUWMM produces the highest runoff, both with respect to peak flow and volume. The SWMM and FSP appear to produce the same volume of runoff, but the FSP has a much lower and earlier peak discharge and a longer recession curve. Effects of these differences for real catchment simulations are shown in a later section.

Hypothetical Pipe Data Tests

Hypothetical pipe data tests were performed for two different pipe diameters and three invert slopes. In addition, two types of upstream and downstream boundary conditions were specified. The first assumes free inflow into the upstream end of the pipes and free outfall at the downstream end. The second assumes a storage tank at the upstream end of the pipe and a diversion structure at the downstream end. Four inflow hydrographs each with three inflow quality constituents were specified for the flow and water quality routing. These data represent a total of 48 data combinations.

One example is described here to illustrate model differences. It is for a 0.61 m (2 ft) diameter circular pipe, 3048 m (10,000 ft) long, having a slope of 0.05 percent, and a Manning's roughness coefficient of 0.01. Free inflow and outflow is assumed. A 2-hr triangular inflow hydrograph is used with a peak flow of 0.168 m^3/sec (5.94 cfs) and a constant flow of 0.017 m^3/sec (0.59 cfs) before and after the triangular inflow. The peak flow represents 90 percent of full pipe flow as computed by Manning's equation. Three conservative inflow quality parameters are specified: a constant concentration of 100 mg/l, a 2-hr triangular shape with a peak concentration of 100 mg/l and a constant concentration of 0 mg/l before and after the triangular shape, and an inverted triangular shape with a minimum concentration of 0 mg/l and a constant concentration of 100 mg/l before and after the triangular shape.

The inflow hydrographs and concentrations and the computed outflow hydrographs and concentrations are shown in Figures 6 to 9. The routing was performed with 5-minute time steps. The routed hydrographs of the BUWMM and SWMM agree quite closely during the rising and falling limb (Figure 6). The SWMM, however, appears to assume zero initial conditions although the inflow hydrograph has a finite flow value, and the peak is considerably more rounded than for the BUWMM. Both models use a kinematic wave formulation and Manning's equation for the flow routing, but the BUWMM solves the equations along a forward characteristic which introduces less numerical dispersion than the finite difference scheme of the SWMM. The FSP also assumes initially zero flow conditions, and produces a much flatter routed hydrograph with a much lower and later peak than the other two models. The model uses a storage routing technique with Manning's equation, but assumes linear relationships between depth, flow area, and discharge. The result is a considerable smoothing of the hydrograph.

Water quality computations are shown only for the BUWMM and the SWMM since the FSP does not include water quality simulation. The constant inflow concentration is routed without modification by the BUWMM (Figure 7). The SWMM, on the other hand, assumes zero initial conditions, and computes a rising limb which reaches the constant concentration value at the time of the peak of the hydrograph, then drops below the constant value again, until it finally approaches the constant value 6 hours after the beginning of the simulation period.

Similarly shaped routed concentrations are computed by the BUWMM and the SWMM for the triangular inflow concentrations. The BUWMM, however, computes a higher and earlier peak since its mass routing uses the kinematic wave celerity rather than water flow velocity to facilitate interfacing with the realtime control scheme of the model.

The inverted triangular inflow concentration is slightly modified by the BUWMM quality routing (Figure 9). The SWMM assumes again initially zero concentrations which results in a rising limb until the routed concentration eventually changes into a shape resembling the inflow concentrations. As before, the smoothing is greater and the minimum routed inflow concentration occurs later for the SWMM than for the BUWMM.

It is apparent from the above observations that the BUWMM in general may produce earlier arrival and less smoothing of the routed concentrations. It also appears that the SWMM model has to be run for a period representing the actual flow time through the modeled system prior to the desired simulation period in order to reduce the effects of its inability to read in and consider nonzero initial pipe flow and concentration conditions. Additional comments concerning this deficiency are given for the real catchment data tests.

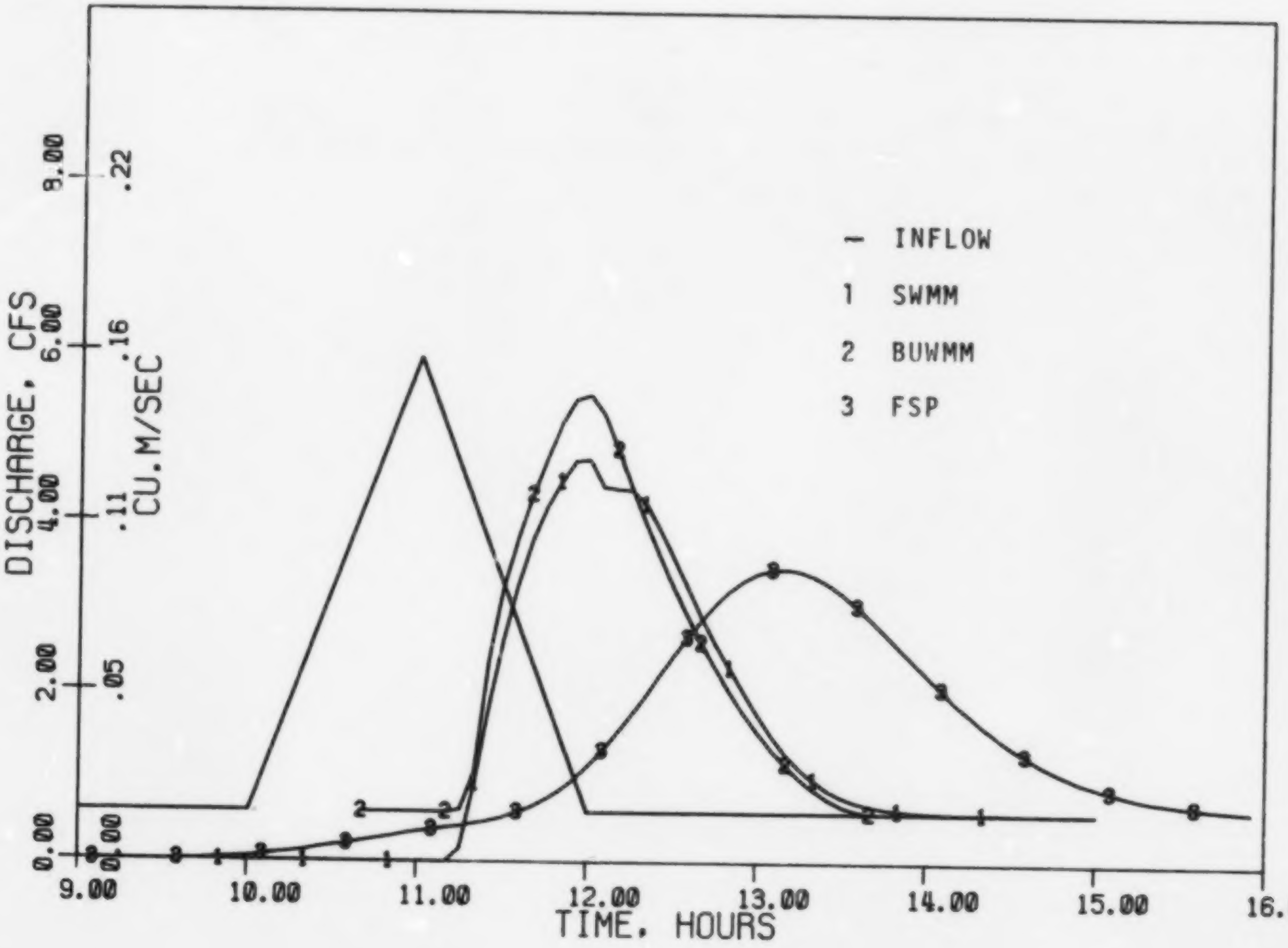

Figure 6. Hypothetical pipe simulation - outflow hydrographs

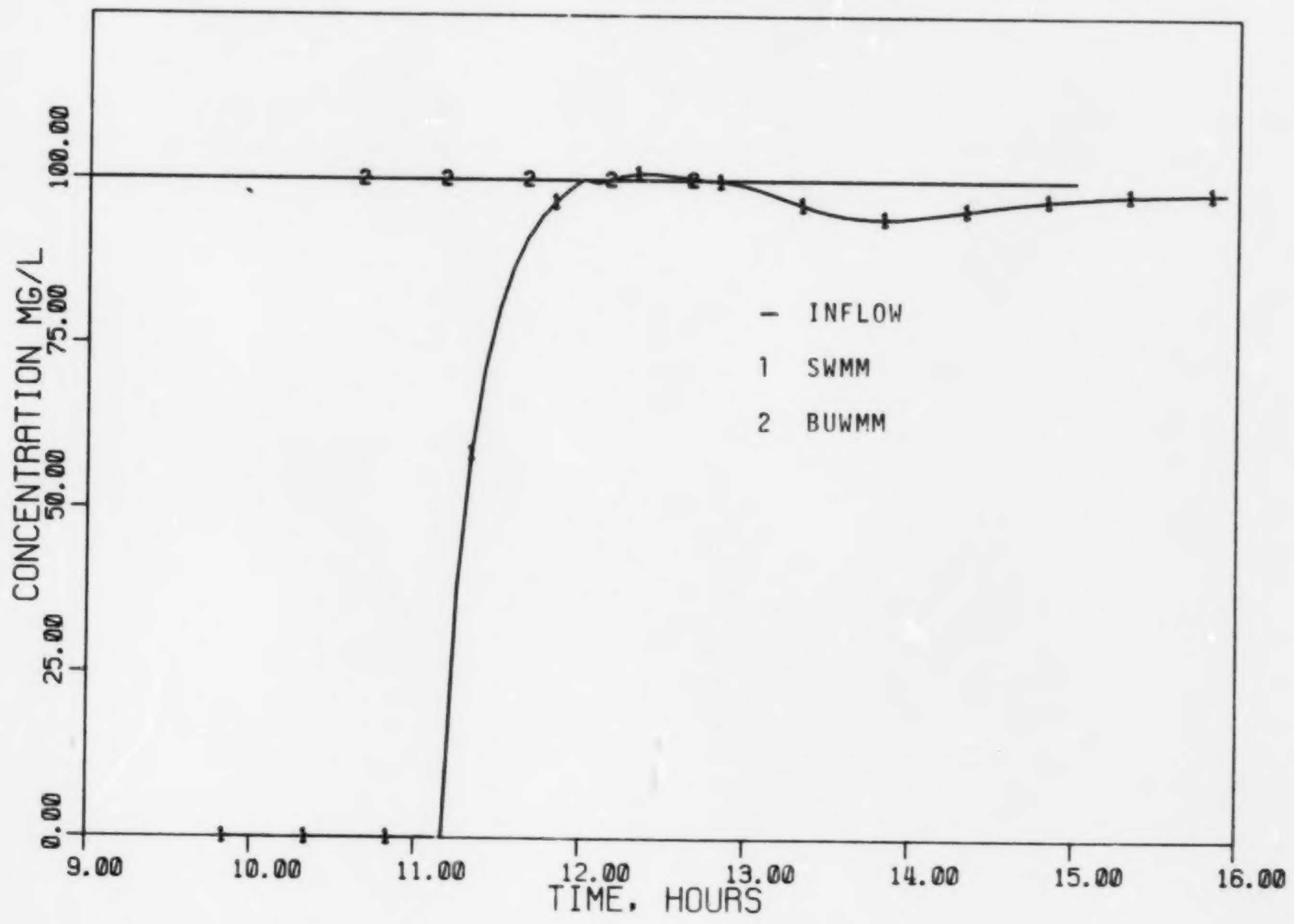

Figure 7. Hypothetical pipe simulation - outflow concentrations for constant inflow concentrations

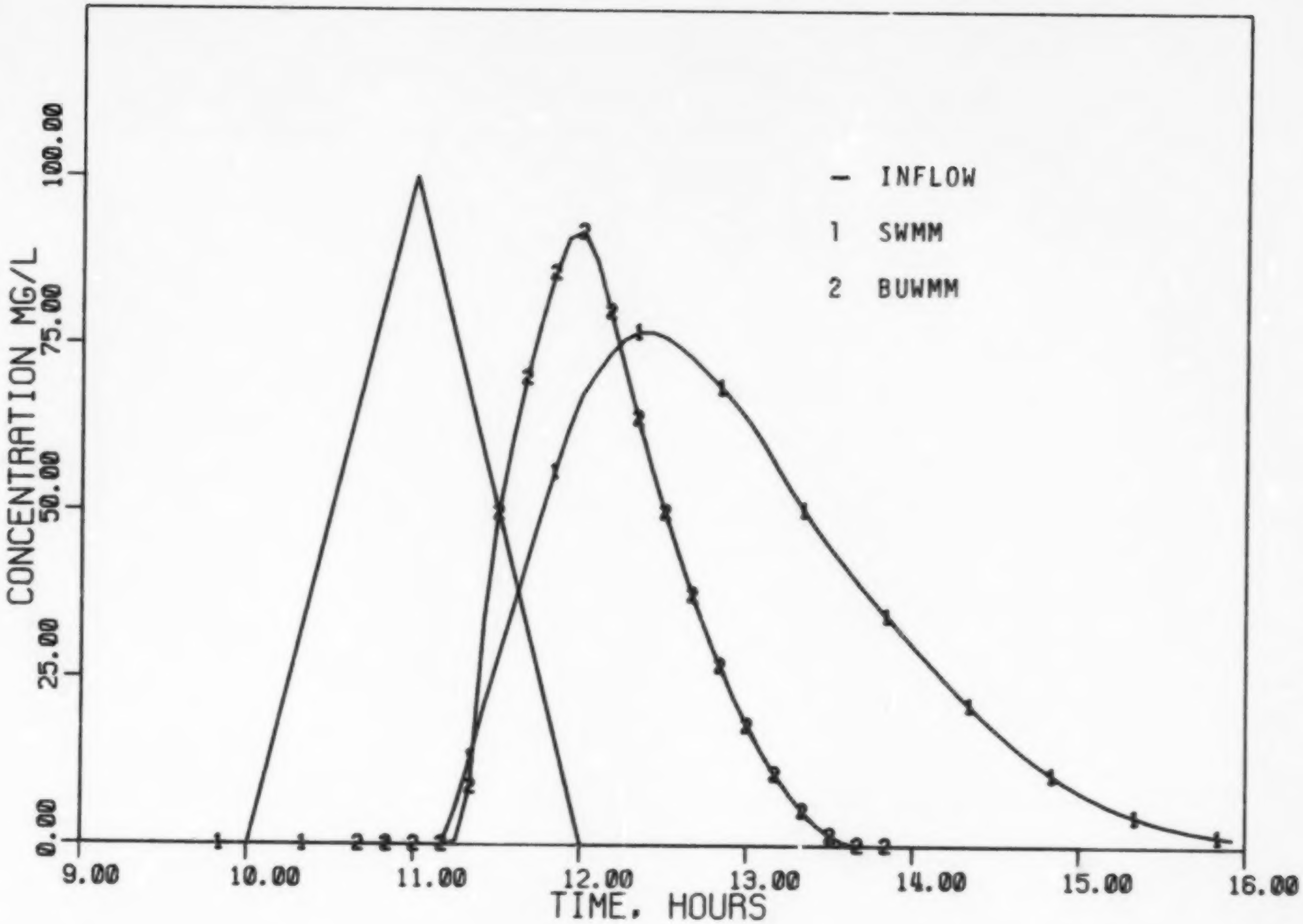

Figure 8. Hypothetical pipe simulation - outflow concentrations for triangular inflow concentrations

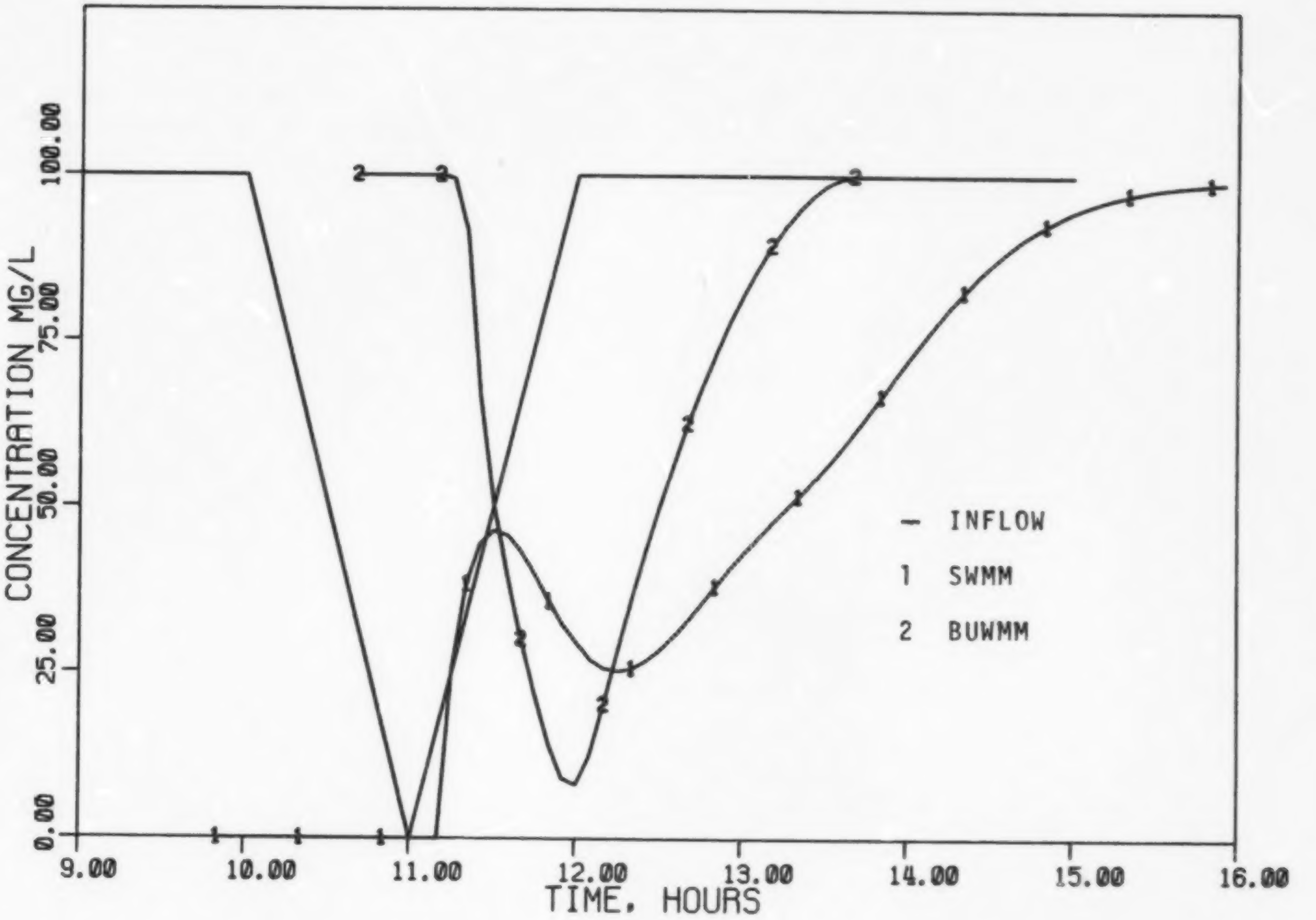

Figure 9. Hypothetical pipe simulation - outflow concentrations for inverted triangular inflow concentrations

Real Catchment Data Tests

An attempt was made to select three real catchments with drainage areas covering three orders of magnitude which have reliable concurrent rainfall, runoff, and water quality data. Since no small catchment was found which had also quality data, the Oakdale Avenue catchment in Chicago, Illinois, with a drainage area of 5.22 ha (12.9 acres) was selected because it has reliable rainfall and runoff data. An intermediate size catchment was selected which had rainfall, runoff, and water quality data, but it was subsequently discovered that the data were unreliable. It was then too late in the study to find substitute data. The Bloody Run catchment in Cincinnati, Ohio, with a drainage area of 964 ha (2380 acres) was selected to represent a large catchment. It has rainfall, runoff, and water quality data. Difficulties were encountered, however, in ascertaining the reliability of the data, and the data had to be scaled from plotted hyetographs, hydrographs, and water quality graphs since only small portions of the measured data were available in tabular form. Only Oakdale catchment simulations are presented here since the Bloody Run simulations were not completed in time for incorporation in this report.

Oakdale Avenue Catchment Simulations

The Oakdale Avenue catchment is located in an urban area about 6 miles northwest of downtown Chicago, Illinois (Tucker, 1968). The Oakdale Avenue catchment is 5.22 ha (12.9 acres) in size (approximately 2-1/2 blocks long by 1 block wide) and consists entirely of residential lots and adjoining streets.

The backbone of the drainage system consists of a 76-cm (30-in.) diameter reinforced concrete combined sewer that drains east along Oakdale Avenue for about two blocks. It drains into a 3.20 m x 3.20 m (10.5 ft x 10.5 ft) concrete combined trunk sewer that drains north toward the North Branch of the Chicago River. The diameters of the lateral sewer pipes vary from 25 cm (10 in.) to 76 cm (30 in.). The pipe slopes vary from 0.30 to 4.20 percent. The Manning coefficients of these pipes were assumed to be 0.012.

As shown in Figure 10, the catchment was divided into 13 subcatchments for the runoff simulations. Each of the 13 subcatchments has its individual inlet manhole. The subcatchments vary in size from 0.33 ha (0.82 acres) to 0.65 ha (1.60 acres). The imperviousness of the subcatchments varies from 39.5 to 56.5 percent. Ground slopes vary from 0.37 to 0.90 percent. Manning's roughness coefficient was assumed to be 0.012 or 0.013 for the impervious areas and 0.350 for the pervious areas.

The maximum infiltration, the minimum infiltration, and the decay rate of infiltration were assumed to be 63.5 mm/hr (2.50 in/hr), 11.4 mm/hr (0.45 in/hr), and 0.00115 sec^{-1}, respectively. The overall volumes of retention storage on the pervious and the impervious areas were assumed to be 5.08 mm (0.20 in.) and 2.03 mm (0.08 in.), respectively.

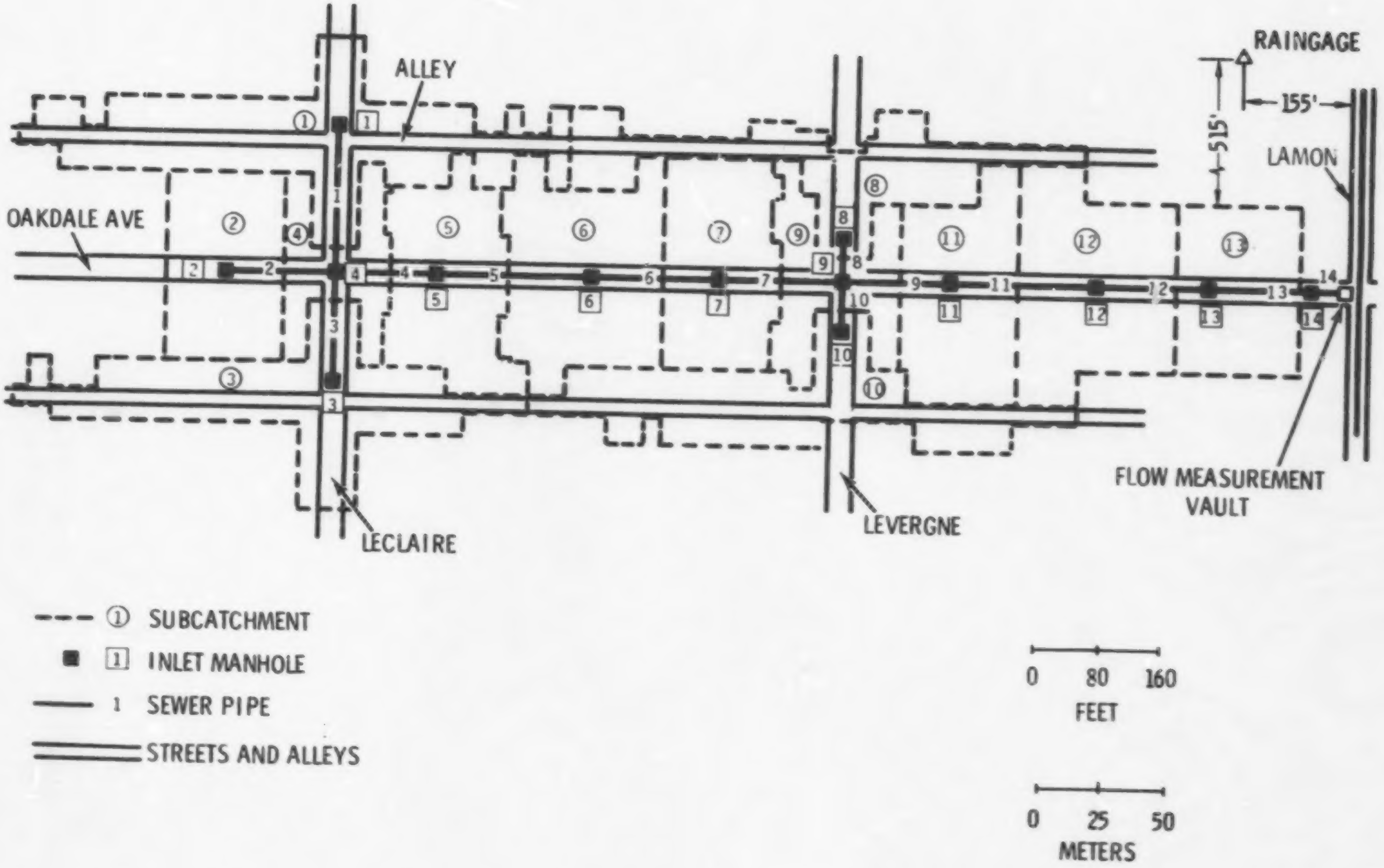
RAINGAGE
155'
515'
LAMON
ALLEY
OAKDALE AVE
LECLAIRE
LEVERGNE
FLOW MEASUREMENT VAULT
① SUBCATCHMENT
1 INLET MANHOLE
1 SEWER PIPE
STREETS AND ALLEYS
0 80 160
FEET
0 25 50
METERS

Figure 10. Map of Oakdale Avenue catchment showing subcatchments and sewer system elements

Rainfall and runoff in the Oakdale Avenue catchment have been periodically measured and recorded since 1957 by the Chicago Department of Public Works, Bureau of Engineering. Detailed rainfall and runoff data for the monitored storms have been reported by Tucker (1968). Runoff has been measured with a Simplex 76-cm (30-in.) type "S" parabolic flume housed in a vault located on the corner of Lamon and Oakdale Avenues. Rainfall measurements have been conducted using a tipping bucket raingauge which is located about one block north of the drainage area on top of Falconer Elementary School. No runoff water quality measurements were conducted.

In the present study, the rainstorms on the following four dates were selected to test several methods of computing urban runoff quantity: May 19, 1959; July 2, 1960; July 26, 1960; and August 2, 1963. These rainstorms were selected since they represent a typical range of possible combinations of rainfall intensities and durations.

The storm of May 19, 1959, had high intensity rainfall with a short duration followed by low intensity rainfall with a long duration (Figures 11 to 14). This storm generated runoff with one good peak. The storm of July 2, 1960, had lighter long-duration rainfall followed by heavier short-duration rainfall (Figure 15 to 18). Runoff produced by this storm had three peaks and the second peak was the largest. Since the flume was flooded at the flow measuring location during the last high-intensity rainfall, the recession curve of the last peak was not recorded. The storm of July 26, 1960, had relatively light long-duration rainfall which resulted in successive small peaks of runoff (Figures 19 to 22). The storm of August 2, 1963, had two high intensity, short-duration rainfalls which produced two successive medium peaks of runoff (Figures 23 to 26).

Figures 11 to 26 show comparisons between the measured runoff and the runoff computed by the Battelle Urban Wastewater Management Model (BUWMM), EPA Storm Water Management Model (SWMM), Chicago Flow Simulation System (FSP), and Dorsch Consult Hydrograph Volume Method (Dorsch). Each computed hydrograph was plotted in a separate figure to better show differences with the measured runoff. Plotting of all computed runoff hydrographs for a given storm in the same figure may provide better comparisons between models, but preliminary plots showed that the lines would be too close together to adequately distinguish between the different model hydrographs. Runoff periods lasting longer than 2 hours and 20 minutes are plotted on more than one figure with a 20-minute overlap between successive figures.

The measured rainfall and runoff data were reported at 1-minute time intervals and the same interval was chosen for the SWMM and Dorsch model simulations. Five-minute time intervals were used for the BUWMM and FSP due to model limitations. The BUWMM is restricted by a minimum time step of 2 minutes and a maximum of 56 time steps. The 5-minute time interval was selected to reduce the need for dividing a long runoff period into too many segments for the simulation. The FSP requires input data and simulation for a minimum of one day and is restricted by a minimum time step of 5 minutes.

Figures 11 to 26 indicate that the BUWMM, SWMM and Dorsch model can be expected to produce satisfactory runoff hydrographs in most cases for small catchments similar to the Oakdale Avenue catchment. The FSP, however, did not produce satisfactory results which confirms the model developers' original intent to use the model only for the continuous simulation of runoff from large, mostly unsewered catchments using 1-hour time steps. FSP formulations are not adequate to accurately simulate runoff from small sewered areas requiring short time steps. The model's assumption of immediate runoff from sewered areas produces peaks which are too high and occur too early. The assumption that all rainfall falling on impervious areas and a constant fraction falling on pervious areas becomes runoff generally produces also a higher volume of runoff than measured.

The use of 5-minute time steps for the BUWMM resulted in smoothing of the peaks and valleys compared to the measured values. Exceptions are the second peaks of the July 2, 1960 (Figure 16b) and August 2, 1963 (Figure 24) storms which show computed peaks and runoff volumes that are higher than the measured values. This appears to be the result of the model's lack of formulations for catchment moisture balance between storms. The model does not compute the recovery of depression storage and infiltration capacity during dry periods, and consequently computed insufficient rainfall losses for the second runoff period of successive storms.

Similar overestimates of the second peaks and runoff volumes are observed for the SWMM for the August 2, 1963 storm (Figure 23) for the same reasons. The second peak and runoff volume of the storm of July 2, 1960 (Figure 15b) were underestimated, however. This storm had to be split into two simulation periods since it was too long for a single simulation run. The underestimate is caused by insufficient knowledge of initial catchment moisture conditions at the beginning of the second simulation period.

The simulations with the BUWMM, SWMM and FSP were performed by Battelle-Northwest, but the Dorsch model results were furnished by Dorsch Consult. Dorsch simulated only the major runoff periods of each storm and split longer runoff periods into separate simulation periods. This generally produced satisfactory results, with the exception of overestimating the second peak and runoff volume of the storm of August 2, 1963 (Figure 26). This again is caused by uncertainties with respect to initial moisture conditions at the beginning of the second runoff period. The model does not include provisions for computing catchment moisture balance during dry periods, with the exception of the recovery of depression storage on pervious areas.

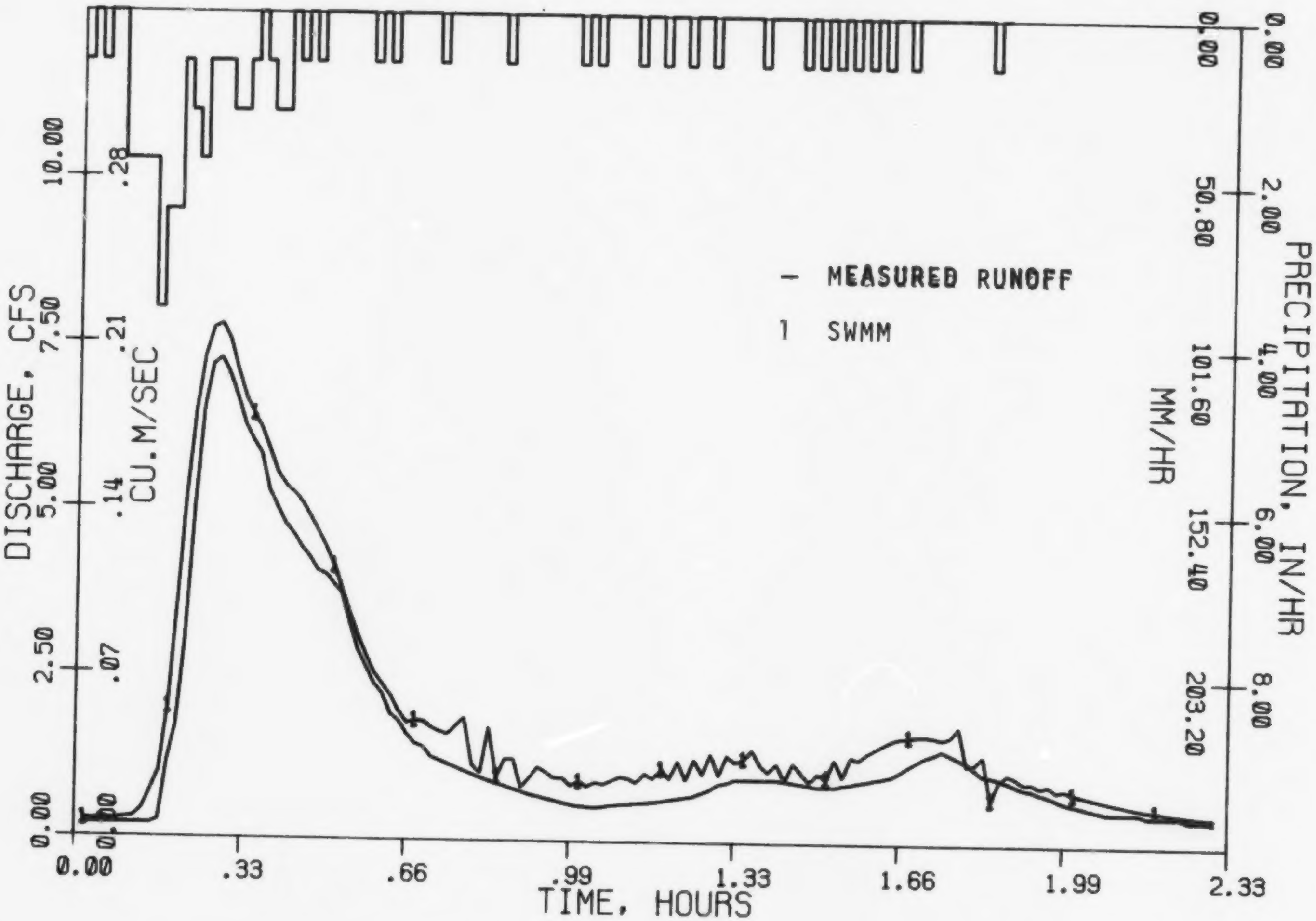

Figure 11. Oakdale storm of May 19, 1959 - EPA Storm Water Management Model

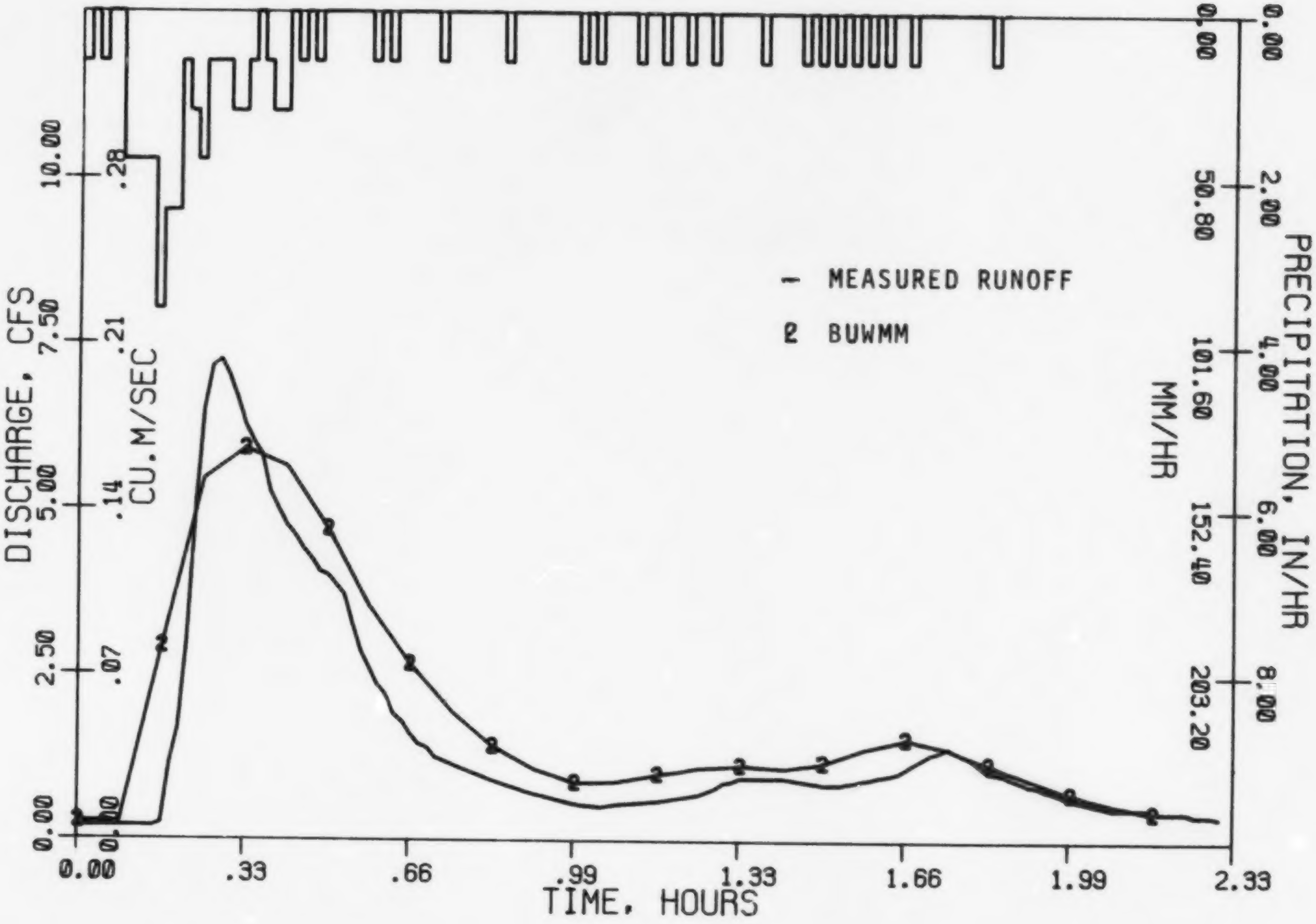

Figure 12. Oakdale storm of May 19, 1959 - Battelle Urban Wastewater Management Model

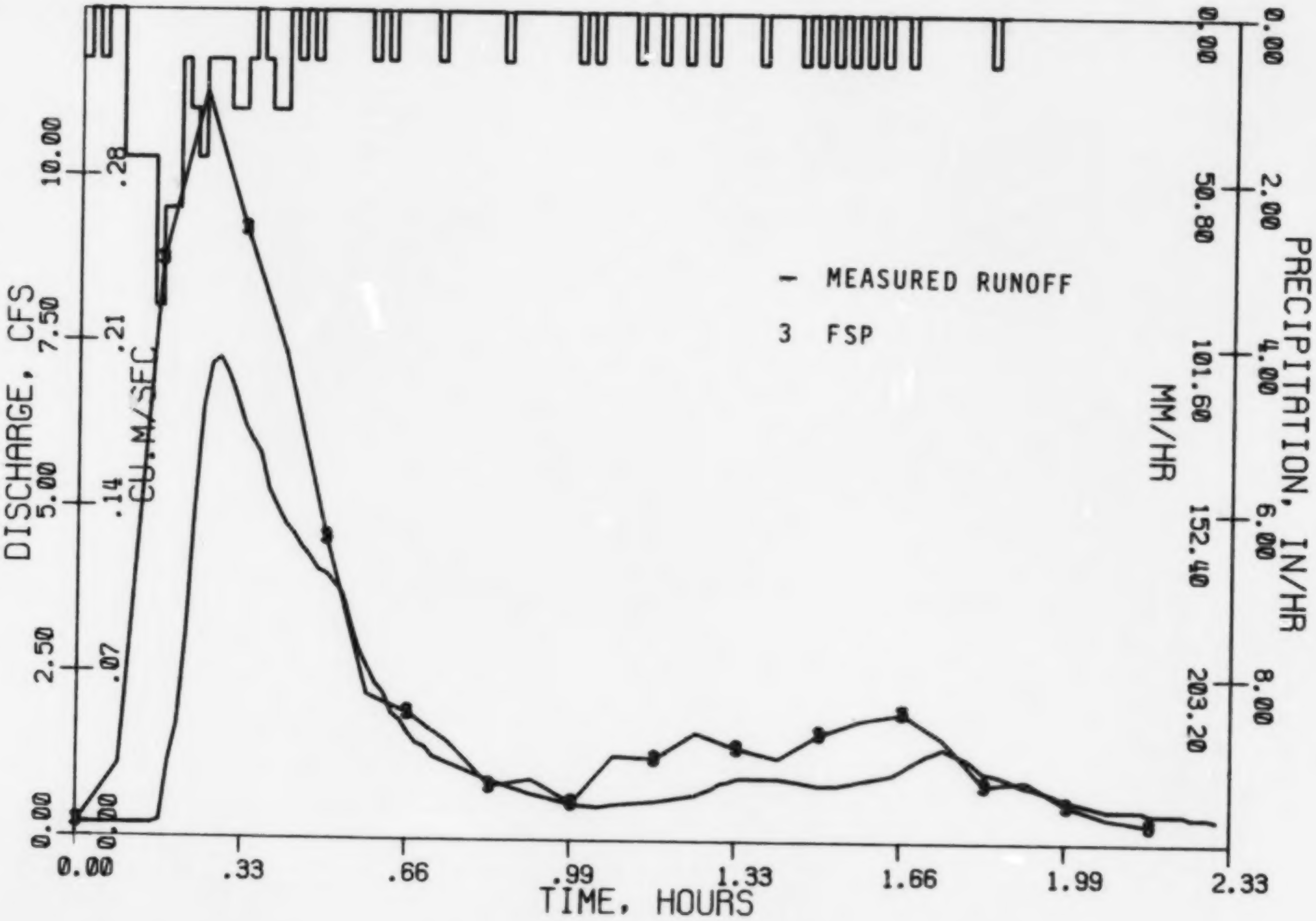

Figure 13. Oakdale storm of May 19, 1959 - Chicago Flow Simulation Program

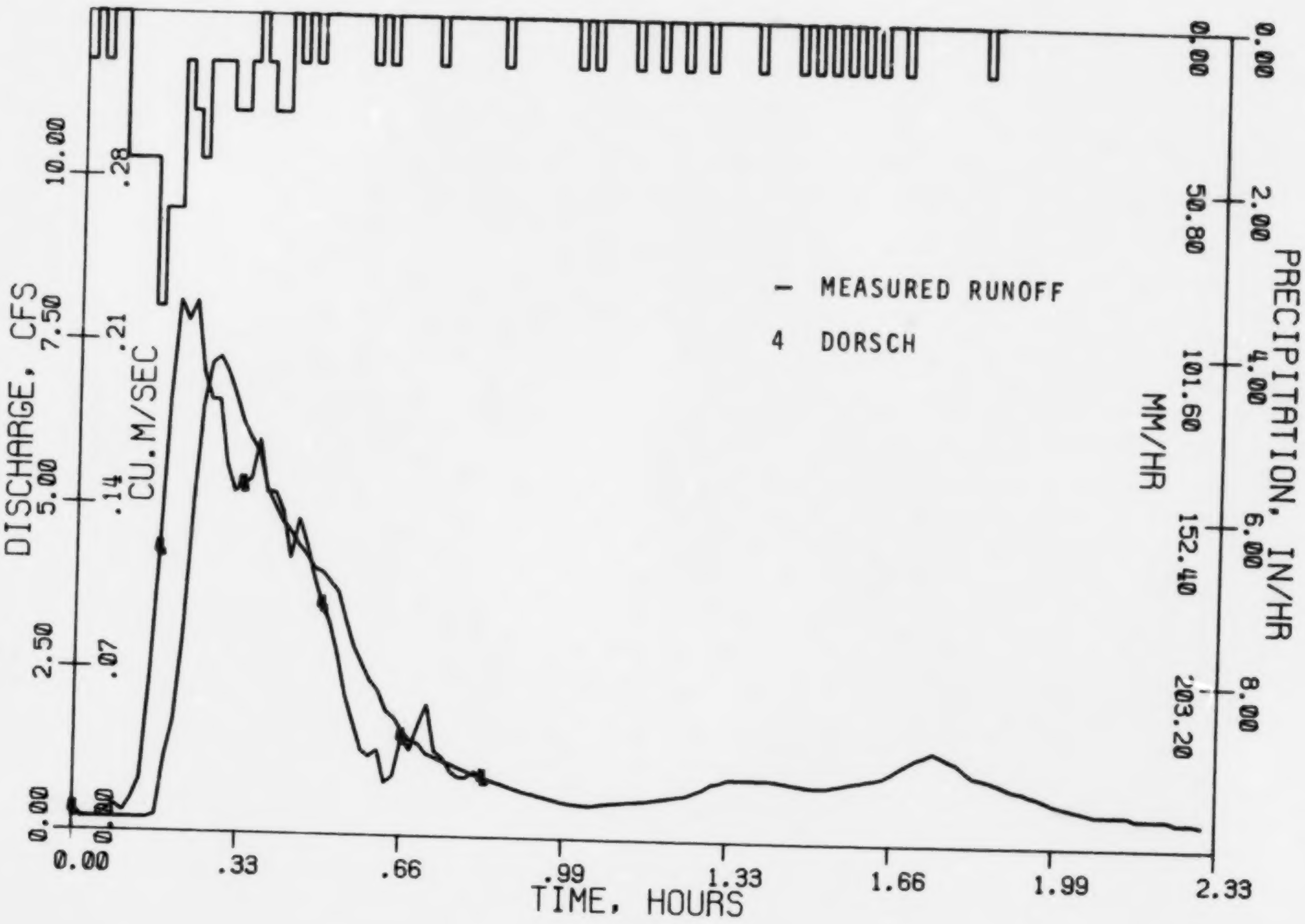

Figure 14. Oakdale storm of May 19, 1959 - Dorsch Hydrograph Volume Method

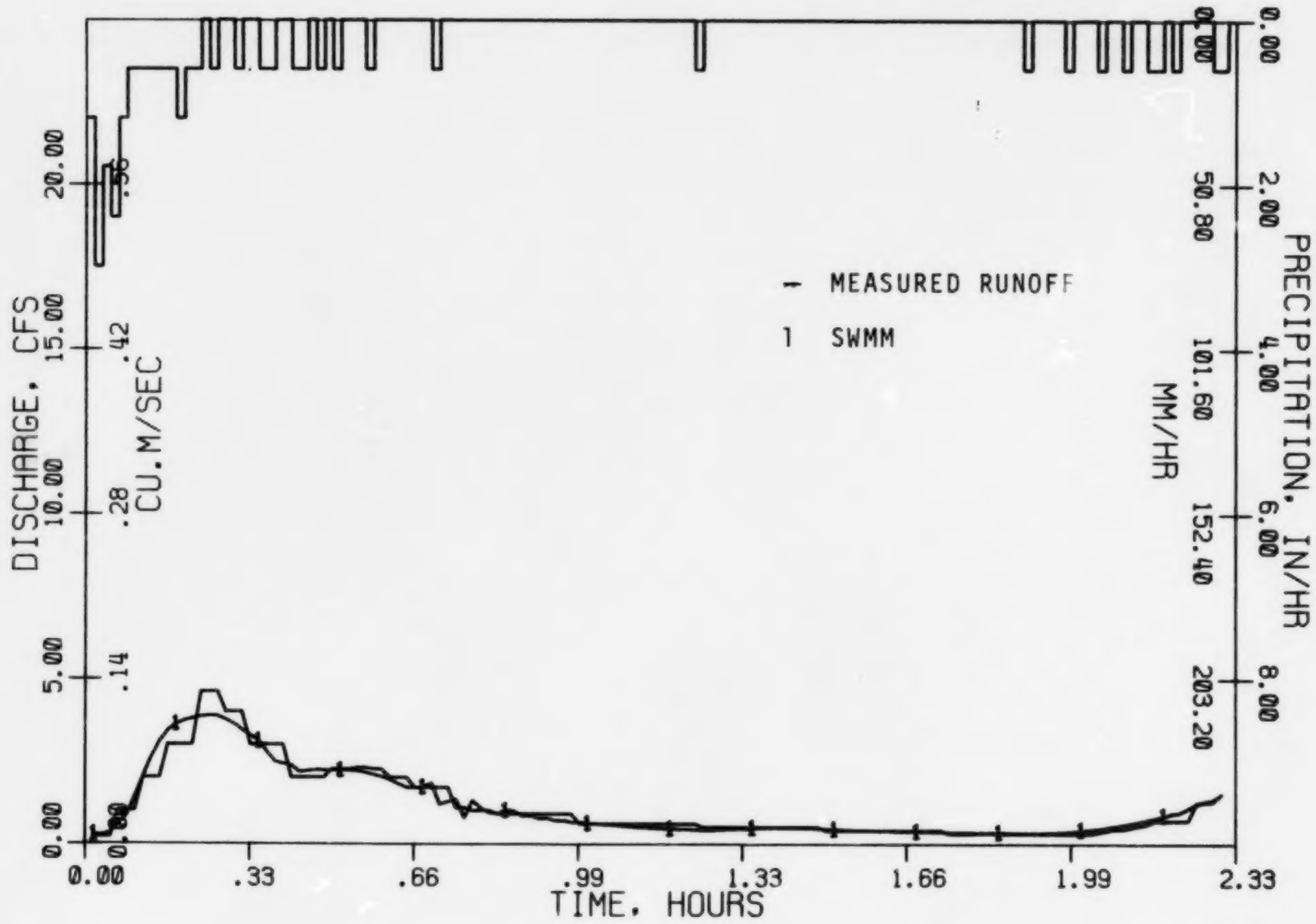

Figure 15a. Oakdale storm of July 2, 1960, part 1 - EPA Storm Water Management Model

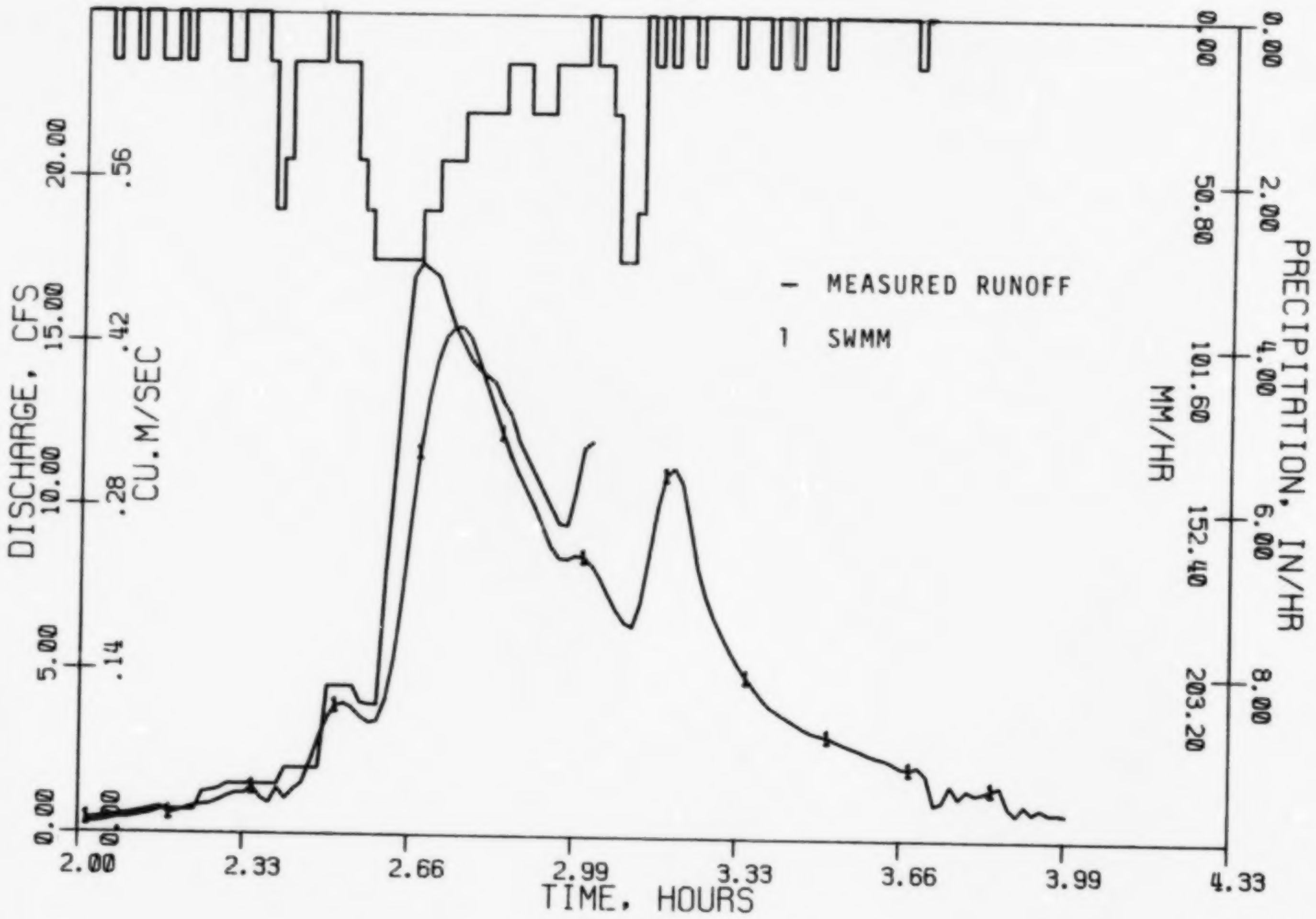

Figure 15b. Oakdale storm of July 2, 1960, part 2 - EPA Storm Water Management Model

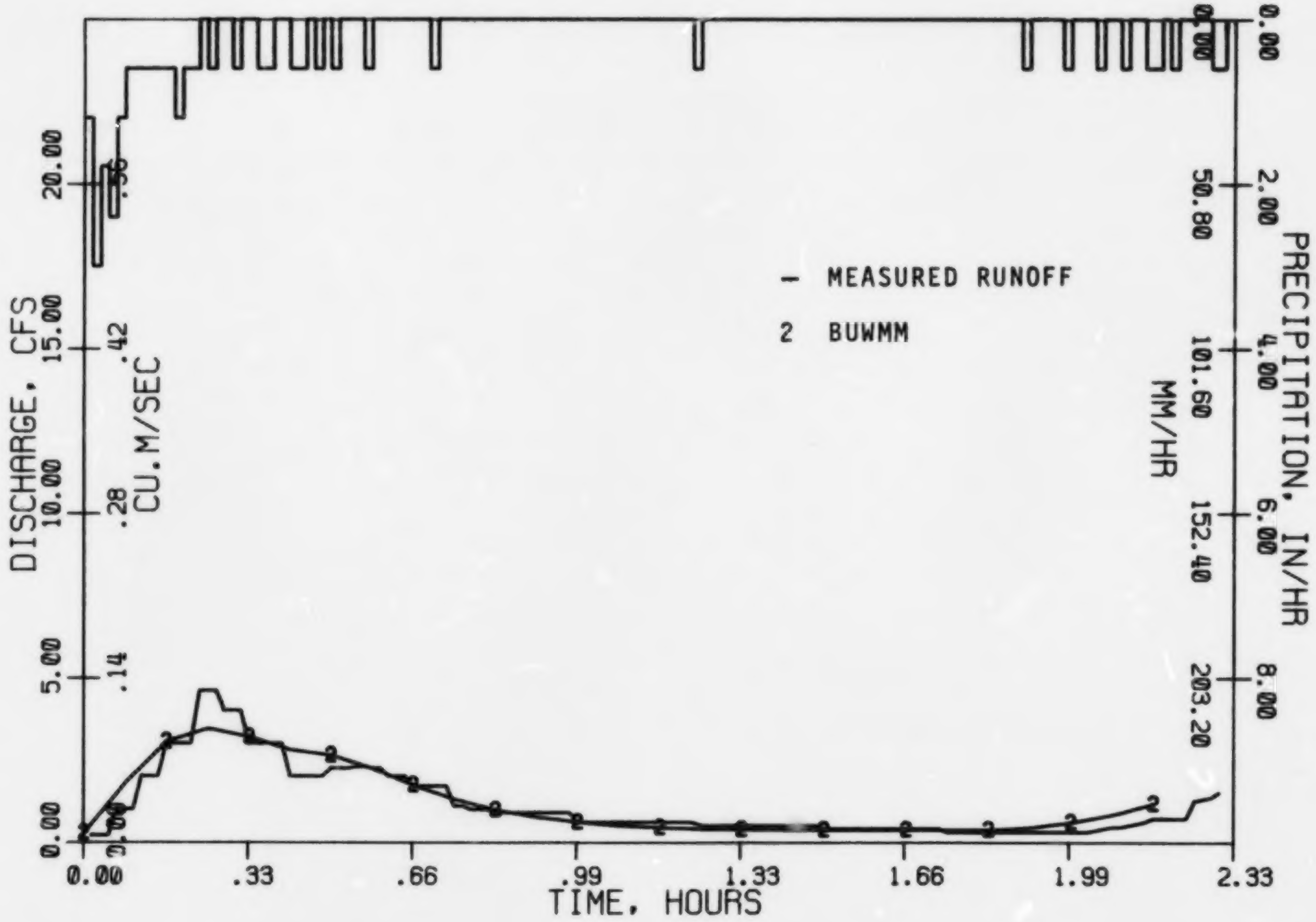

Figure 16a. Oakdale storm of July 2, 1960, part 1 - Battelle Urban Wastewater Management Model

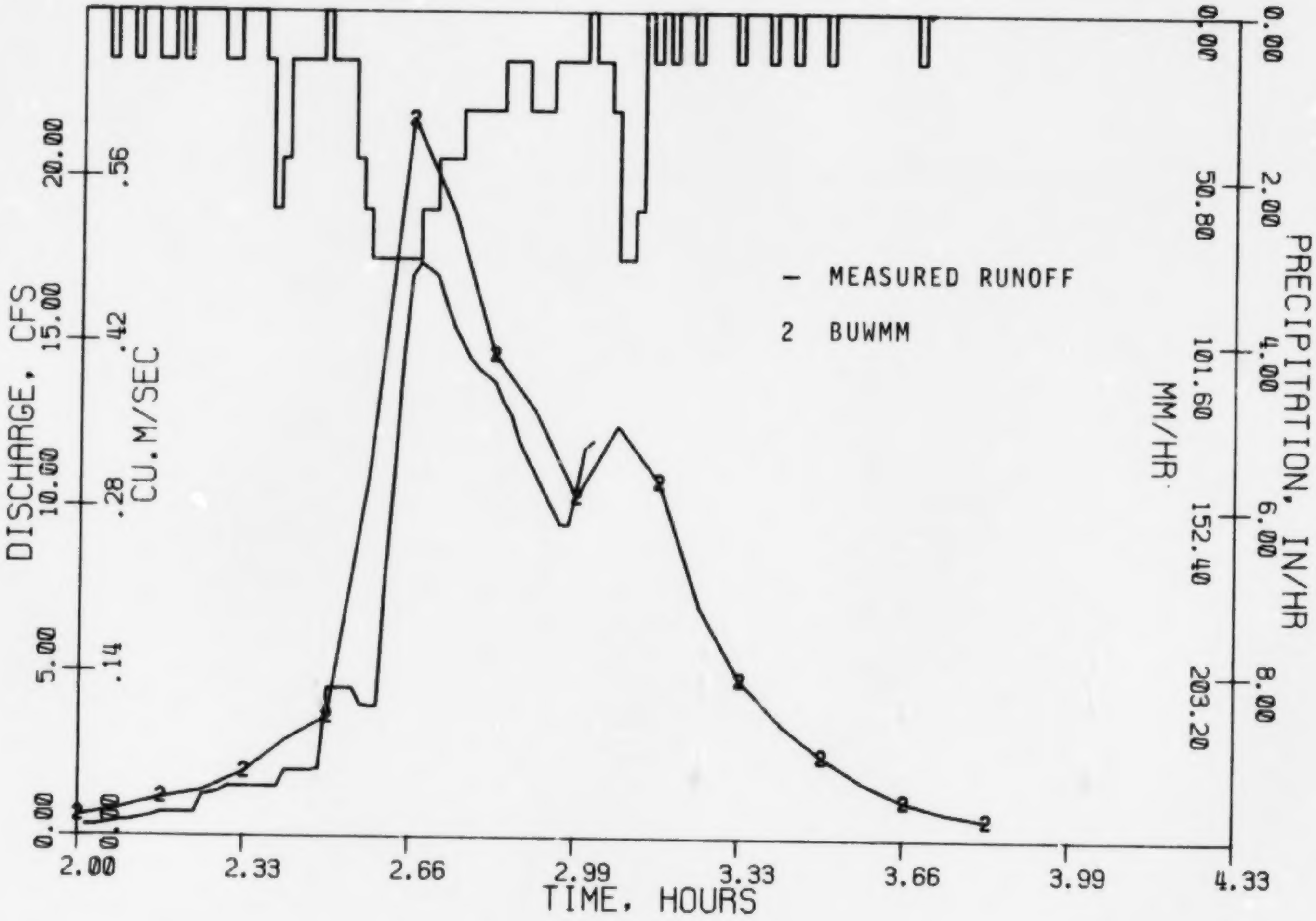

Figure 16b. Oakdale storm of July 2, 1960, part 2 - Battelle Urban Wastewater Management Model

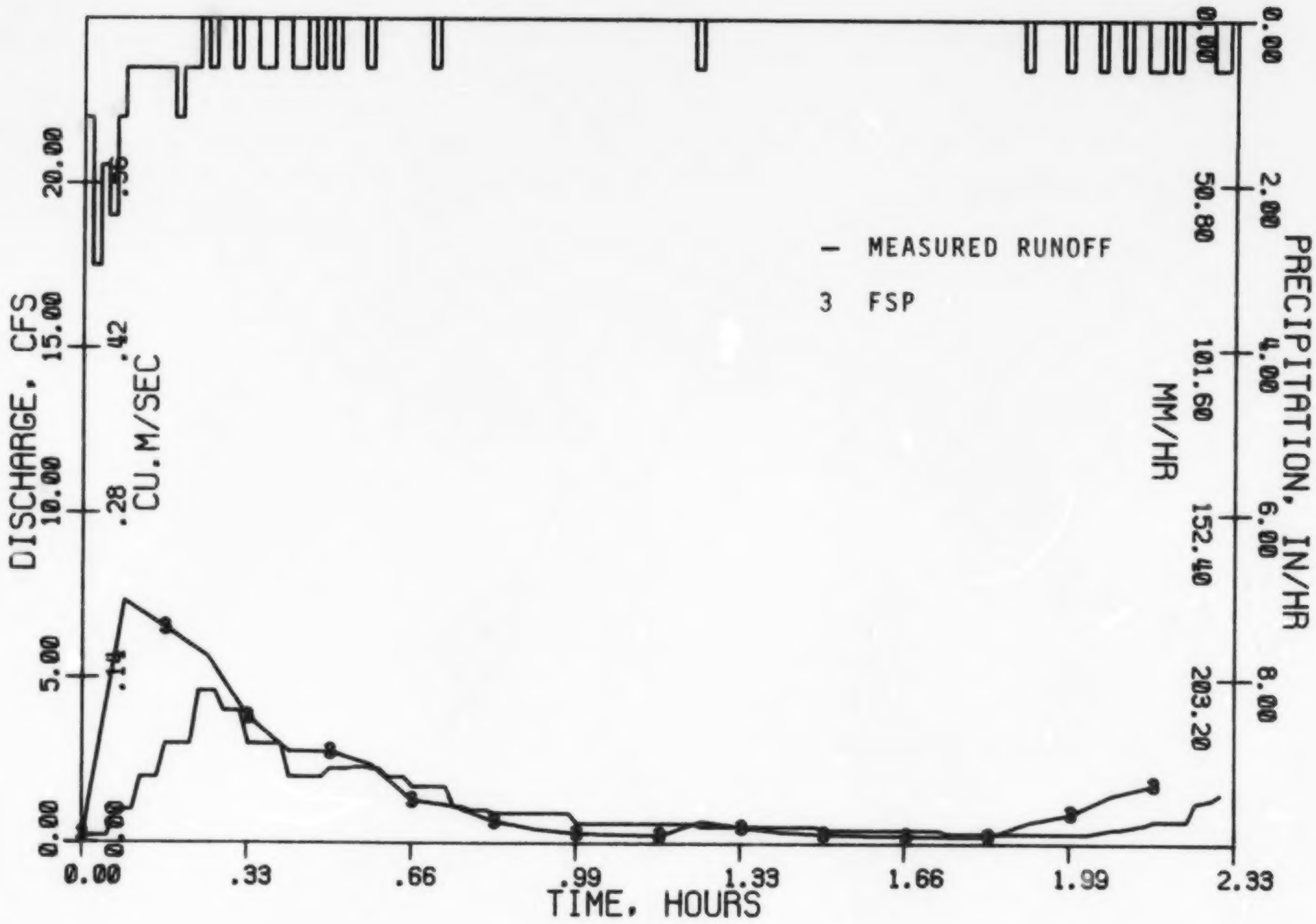

Figure 17a. Oakdale storm of July 2, 1960, part 1 - Chicago Flow Simulation Program

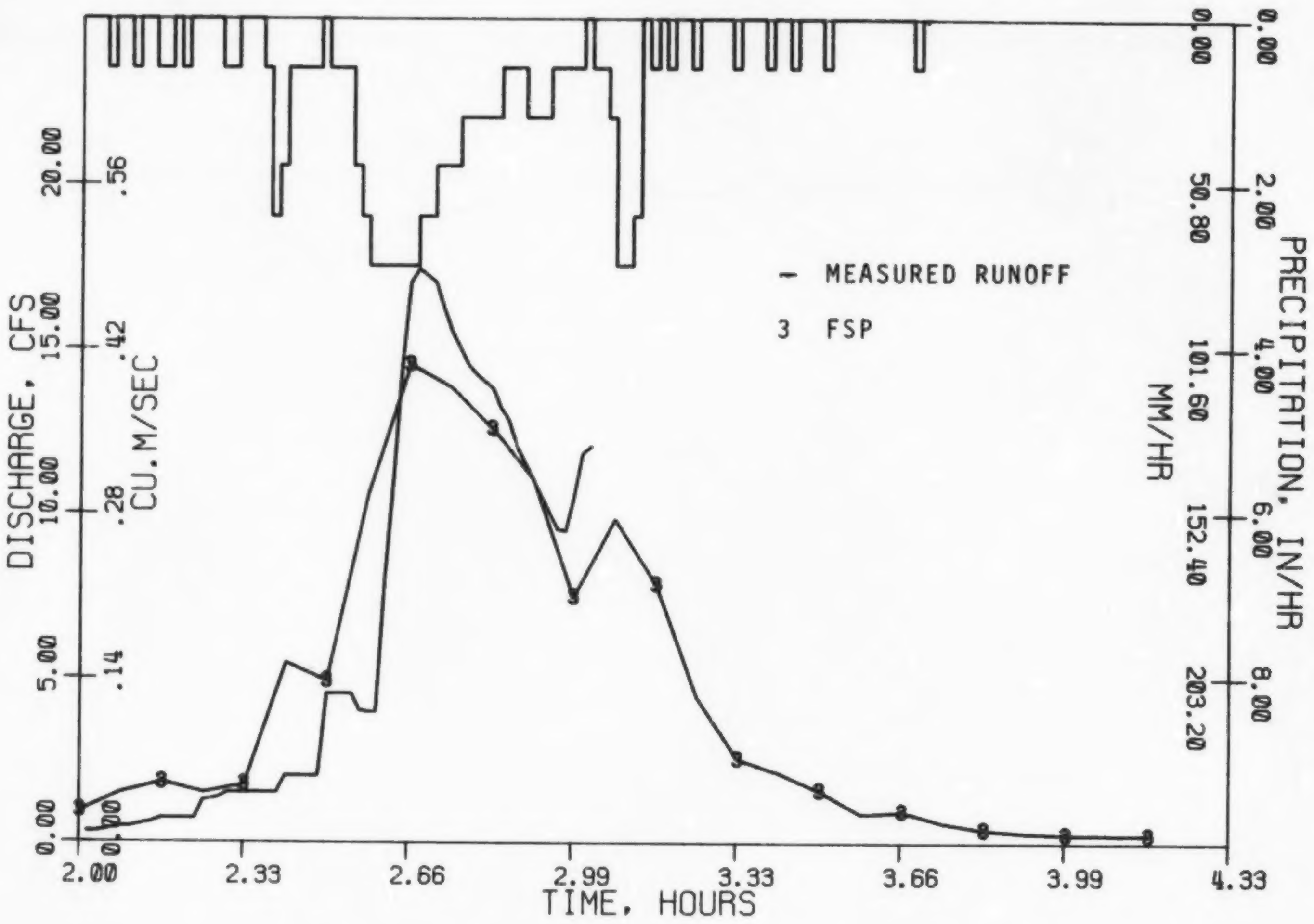

Figure 17b. Oakdale storm of July 2, 1960, part 2 - Chicago Flow Simulation Program

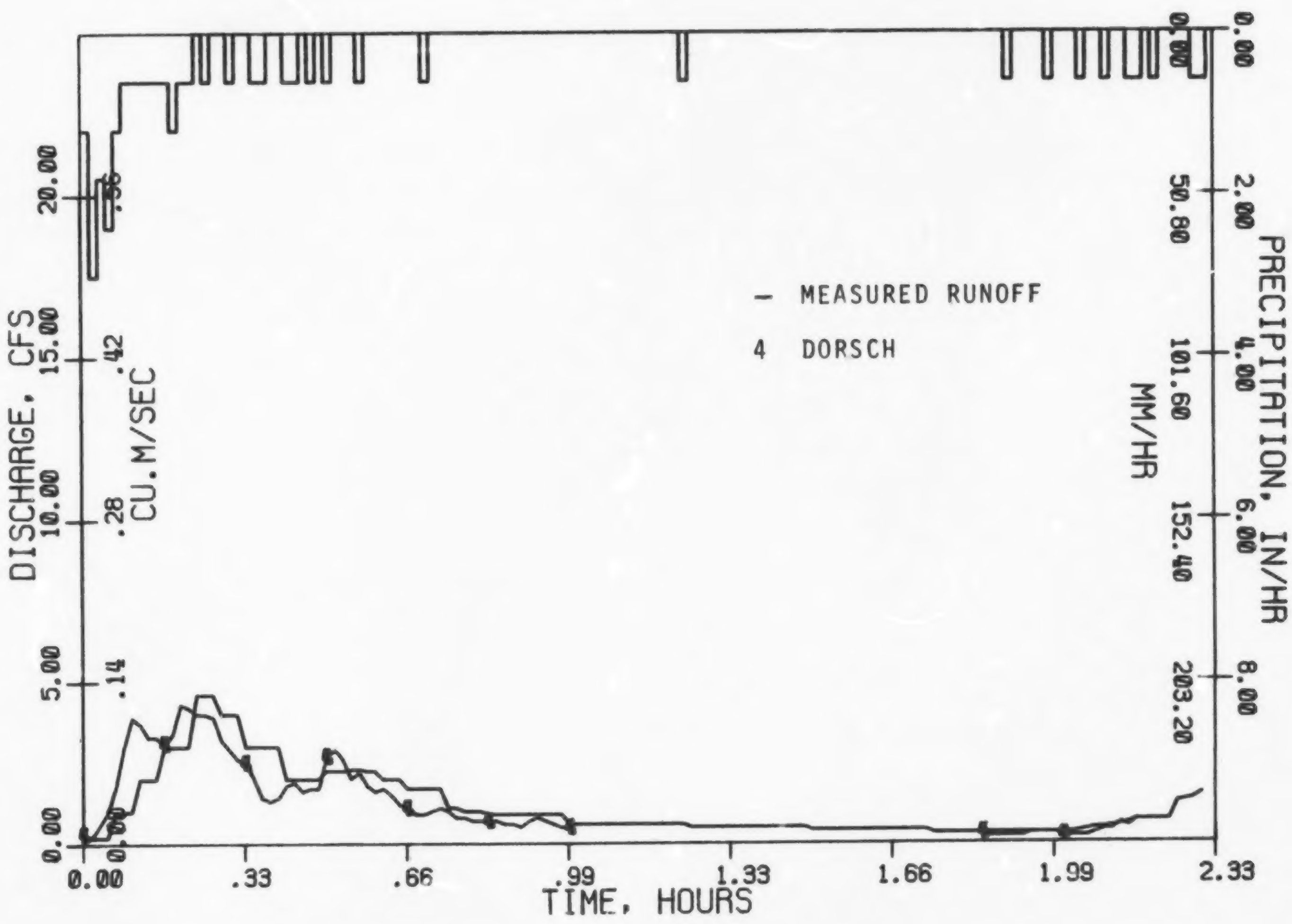

Figure 18a. Oakdale storm of July 2, 1960, part 1 - Dorsch Hydrograph Volume Method

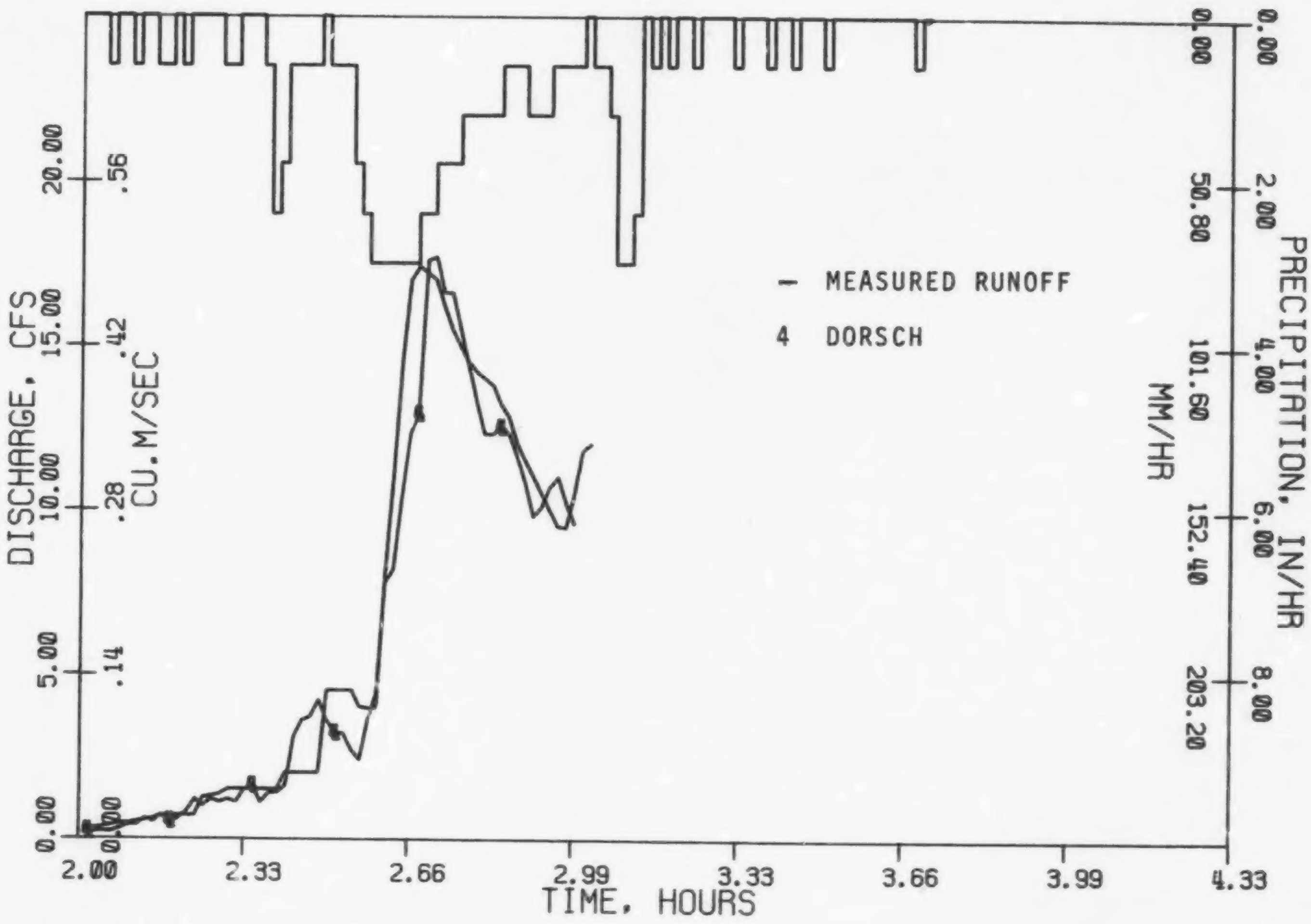

Figure 18b. Oakdale storm of July 2, 1960, part 2 - Dorsch Hydrograph Volume Method

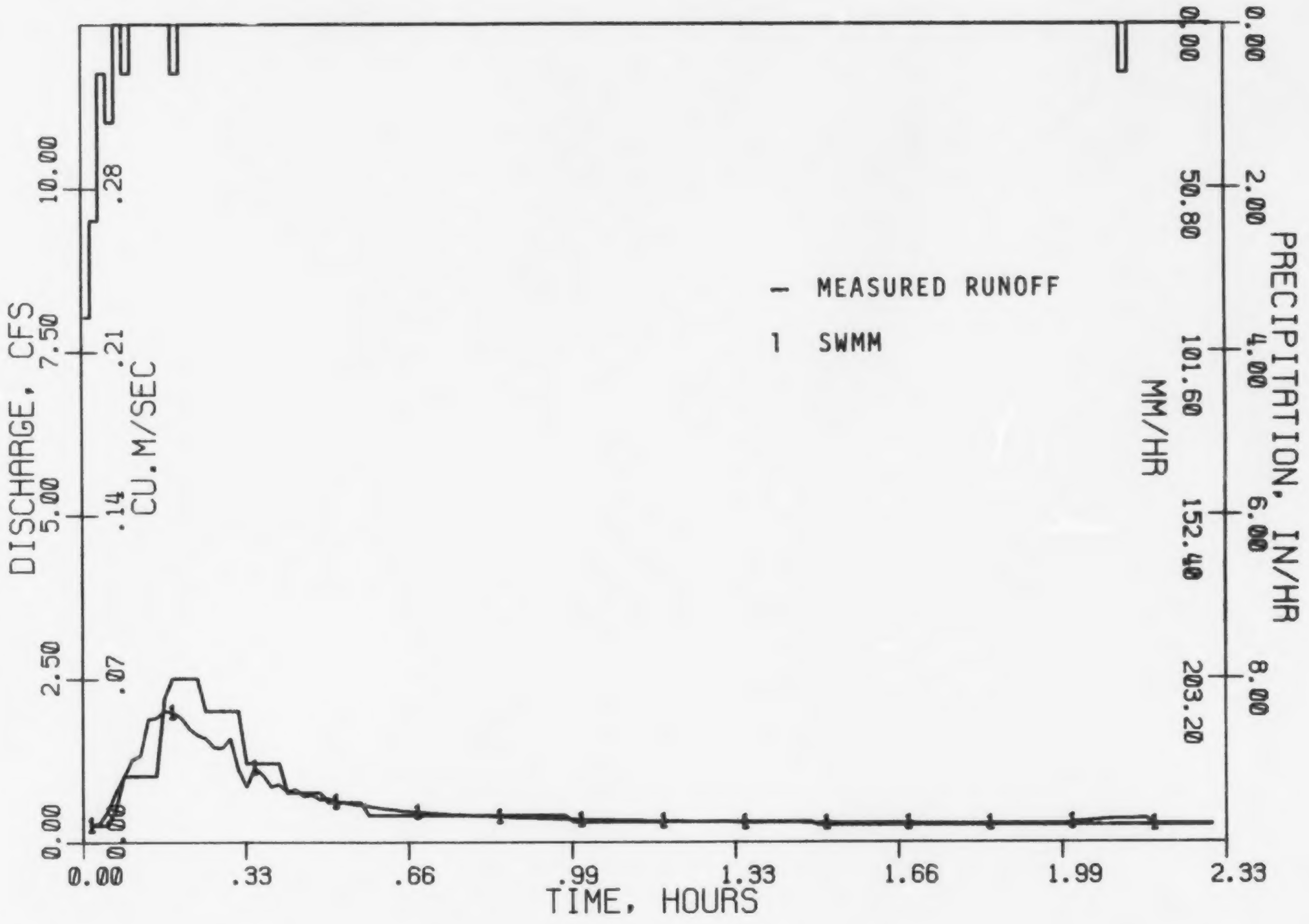

Figure 19a. Oakdale storm of July 26, 1960, part 1 - EPA Storm Water Management Model

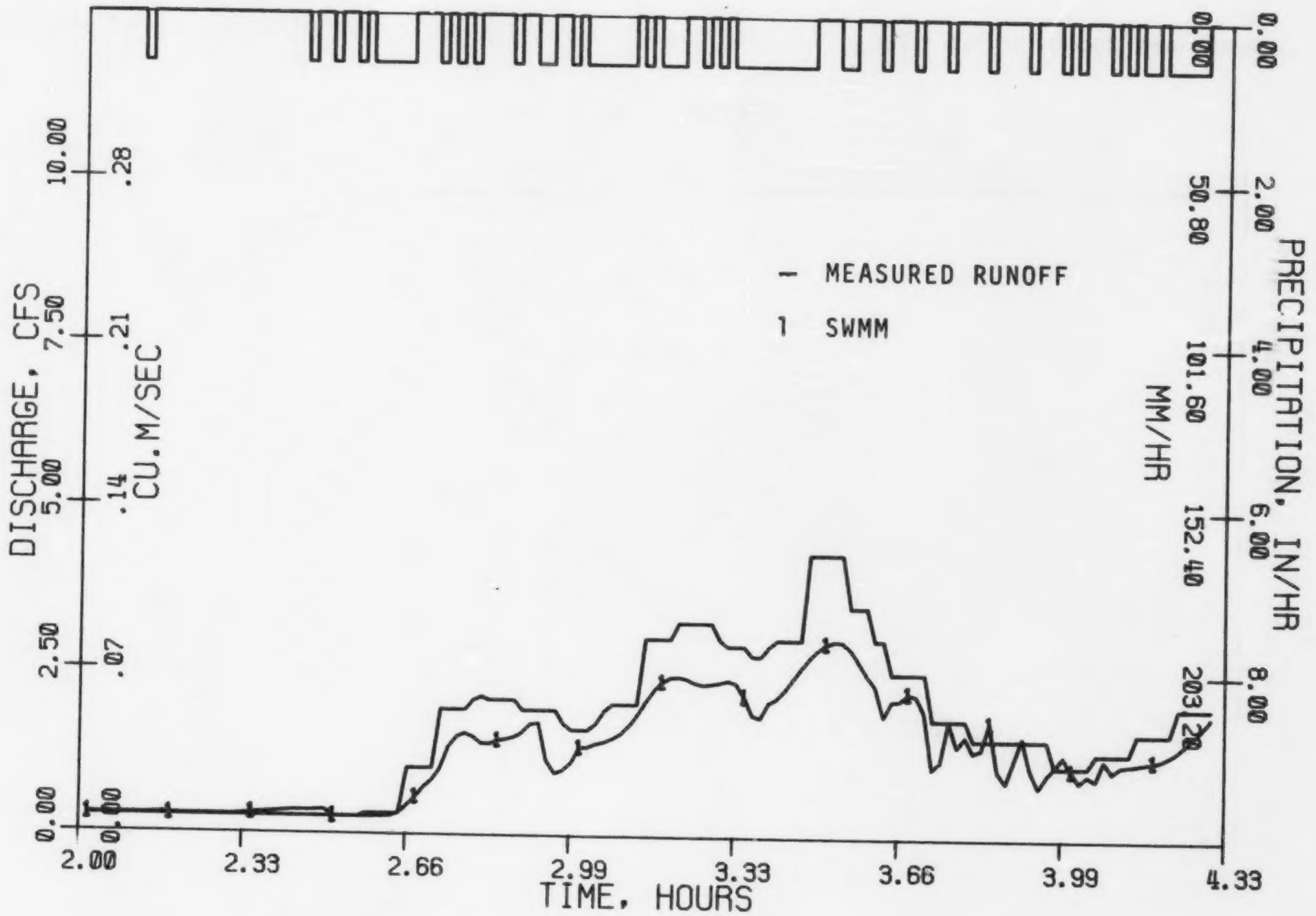

Figure 19b. Oakdale storm of July 26, 1960, part 2 - EPA Storm Water Management Model

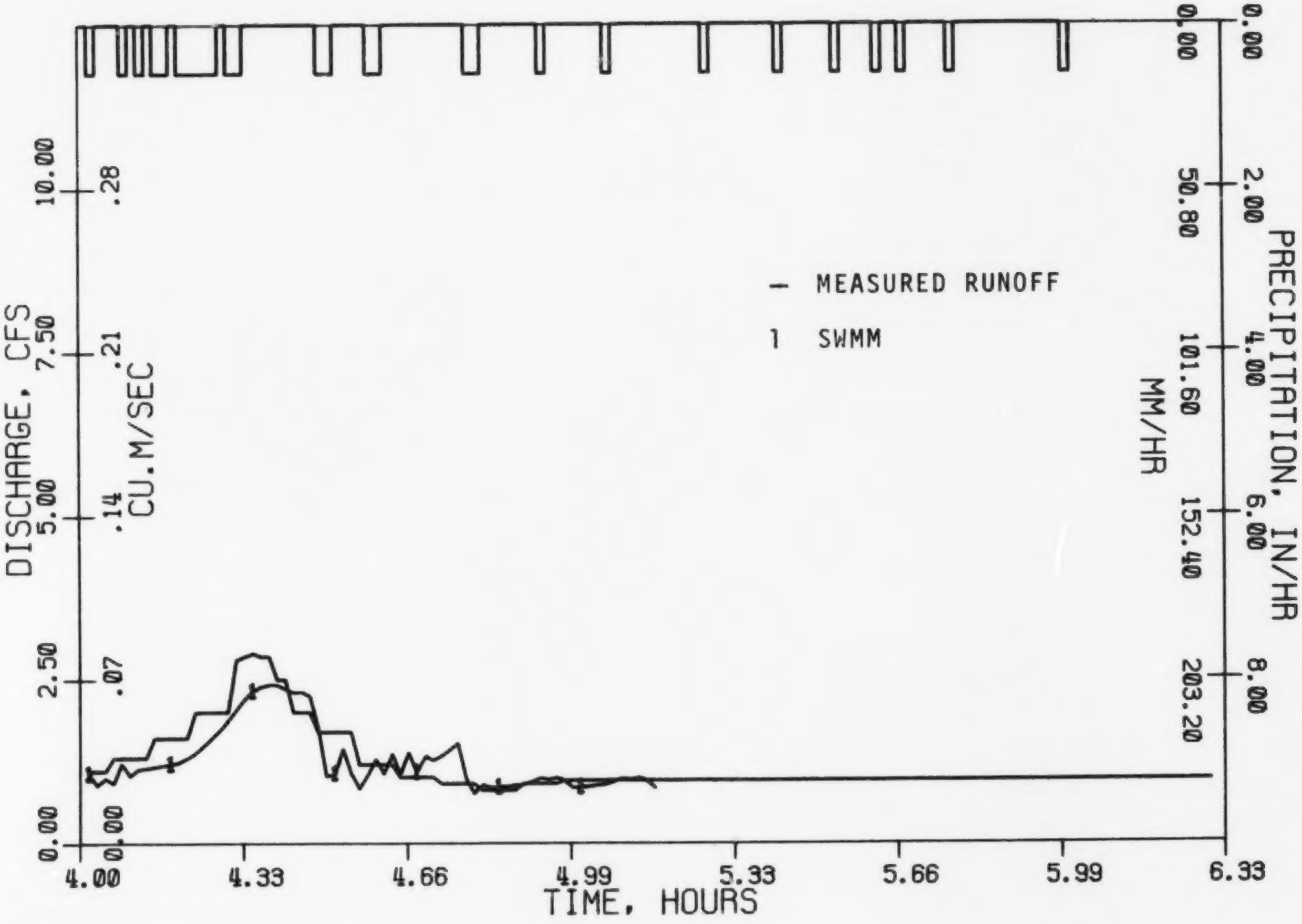

Figure 19c. Oakdale storm of July 26, 1960, part 3 - EPA Storm Water Management Model

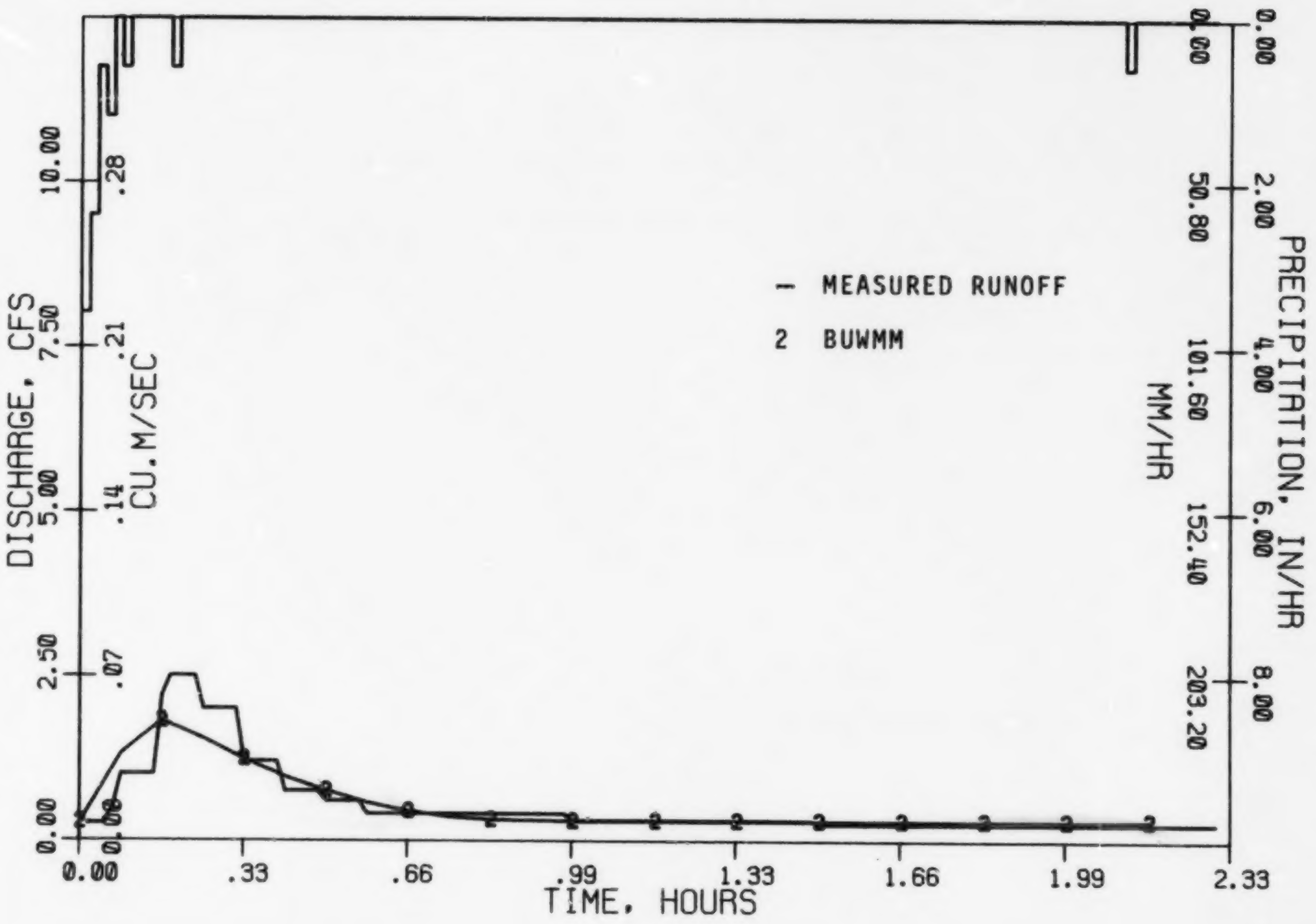

Figure 20a. Oakdale storm of July 26, 1960, part 1 - Battelle Urban Wastewater Management Model

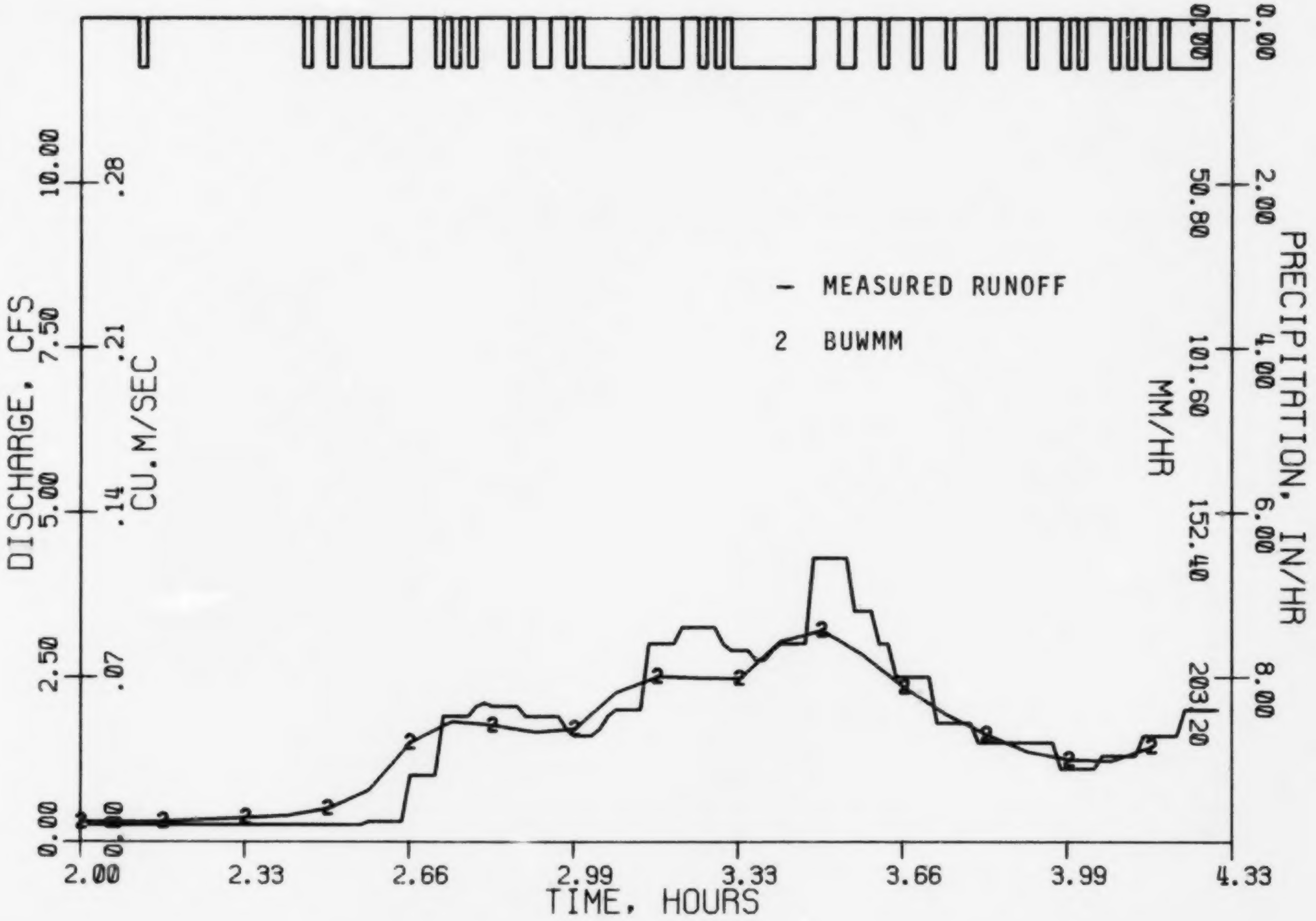

Figure 20b. Oakdale storm of July 26, 1960, part 2 - Battelle Urban Wastewater Management Model

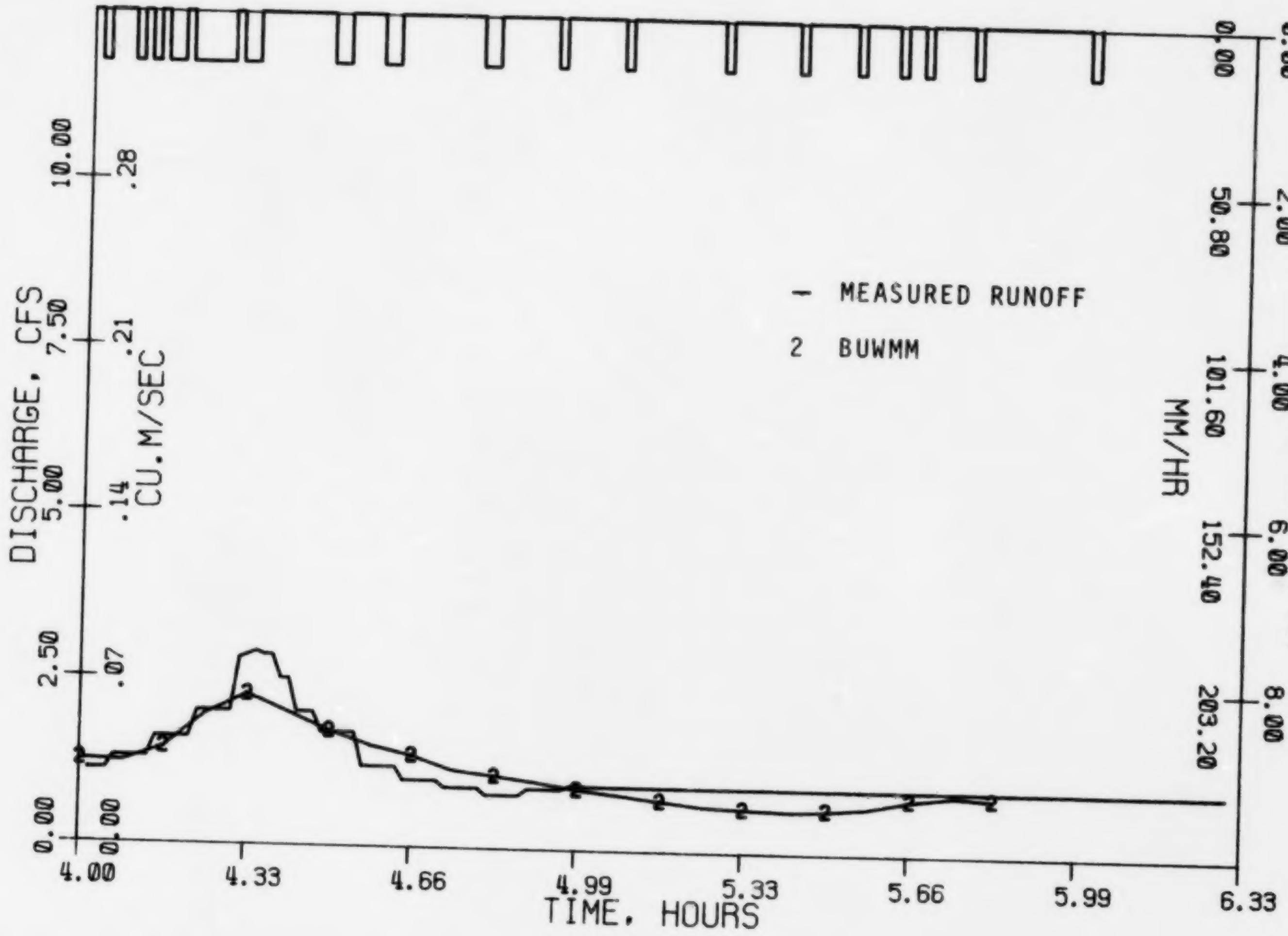

Figure 20c. Oakdale storm of July 26, 1960, part 3 - Battelle Urban Wastewater Management Model

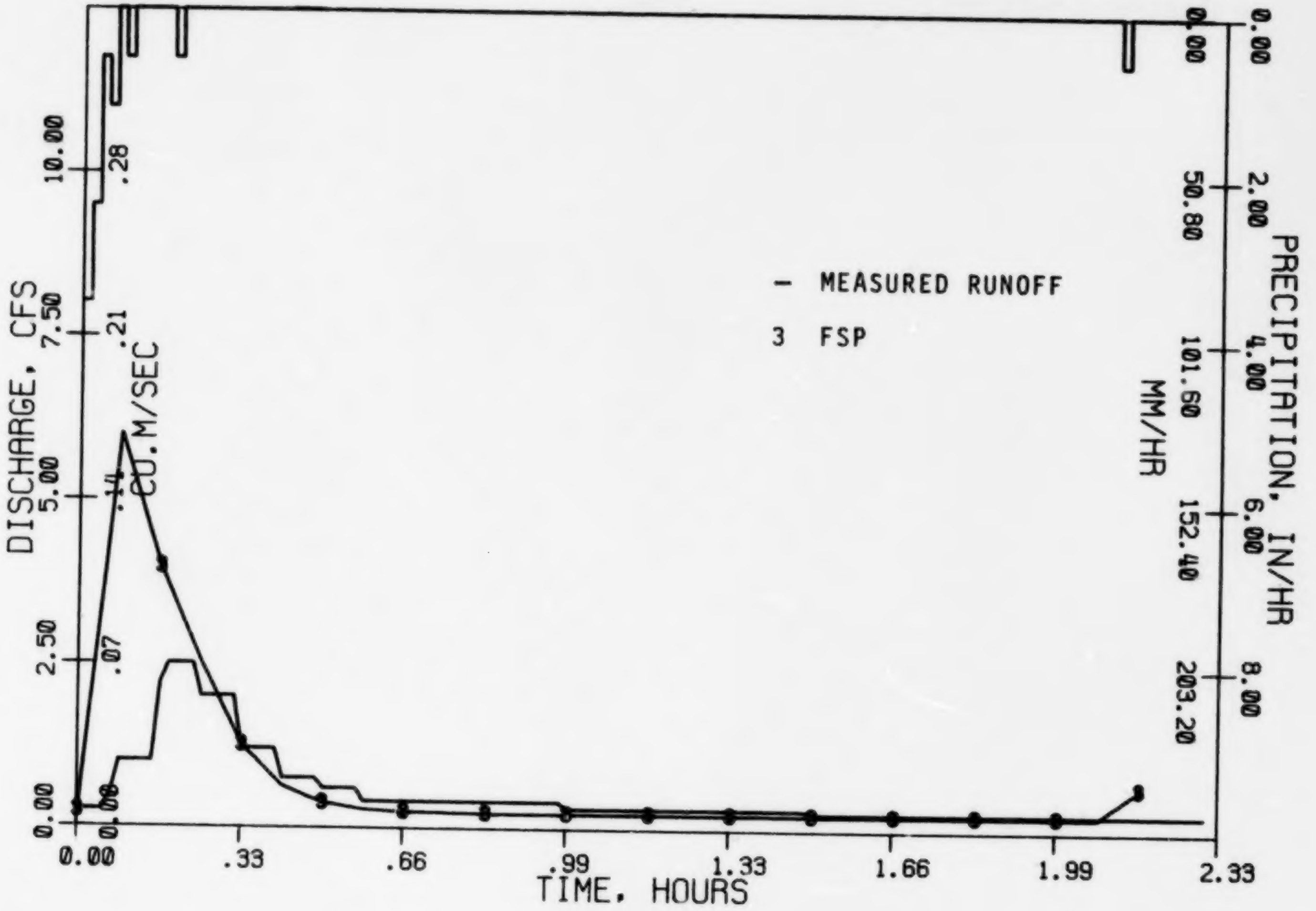

Figure 21a. Oakdale storm of July 26, 1960, part 1 - Chicago Flow Simulation Program

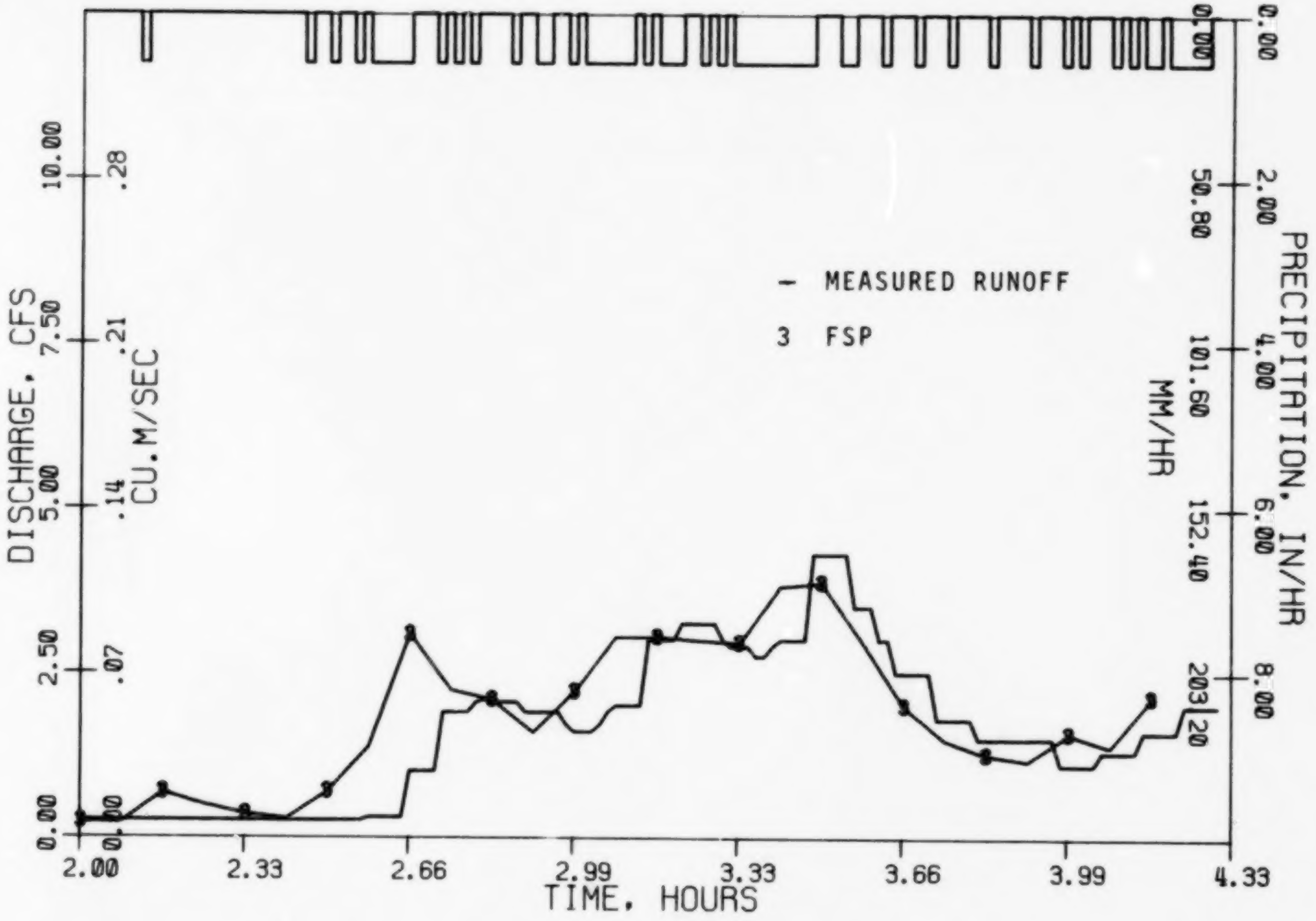

Figure 21b. Oakdale storm of July 26, 1960, part 2 - Chicago Flow Simulation Program

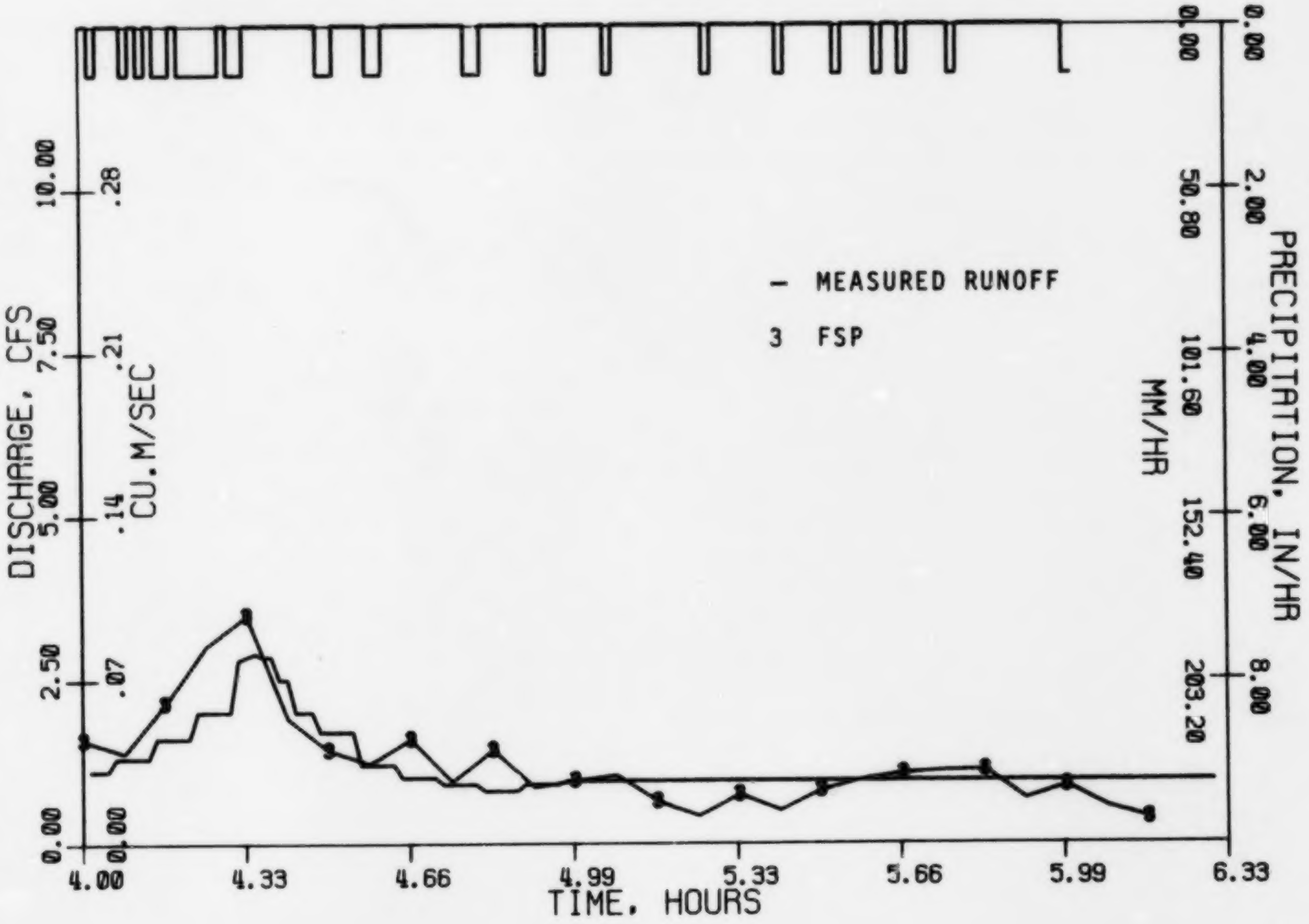

Figure 21c. Oakdale storm of July 26, 1960, part 3 - Chicago Flow Simulation Program

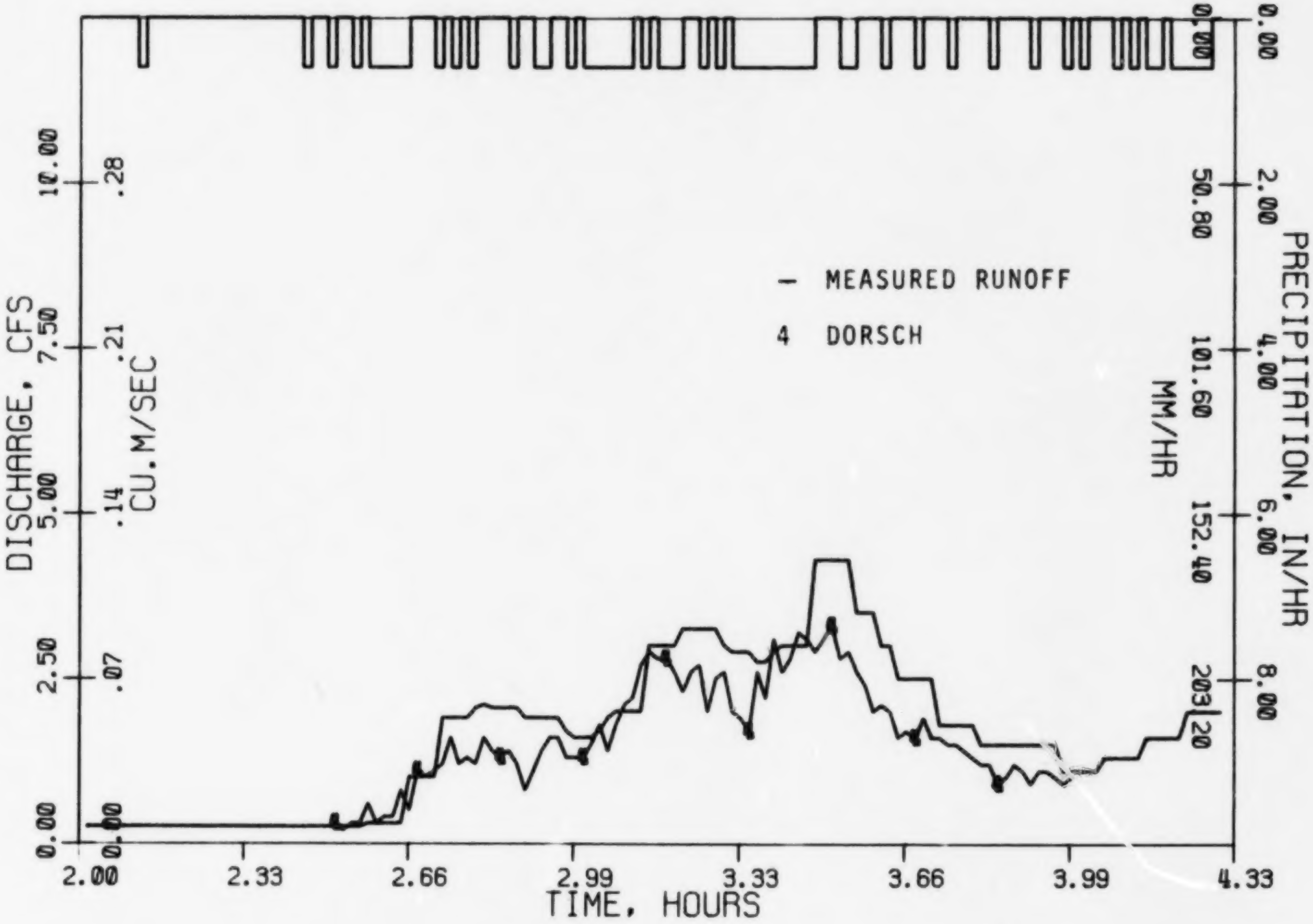

Figure 22a. Oakdale storm of July 26, 1960, part 2 - Dorsch Hydrograph Volume Method (part 1 of storm not simulated by Dorsch)

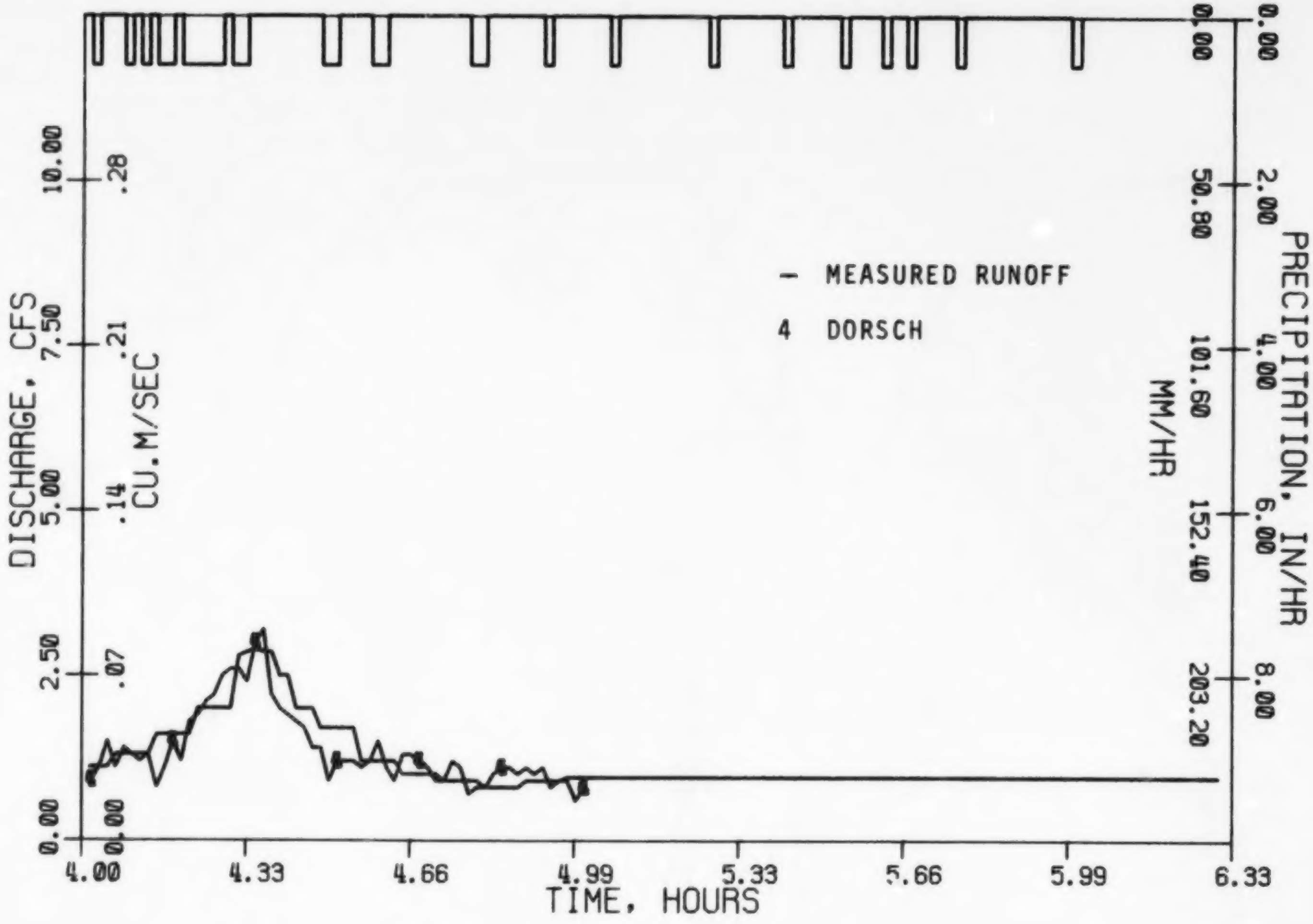

Figure 22b. Oakdale storm of July 26, 1960, part 3 - Dorsch Hydrograph Volume Method

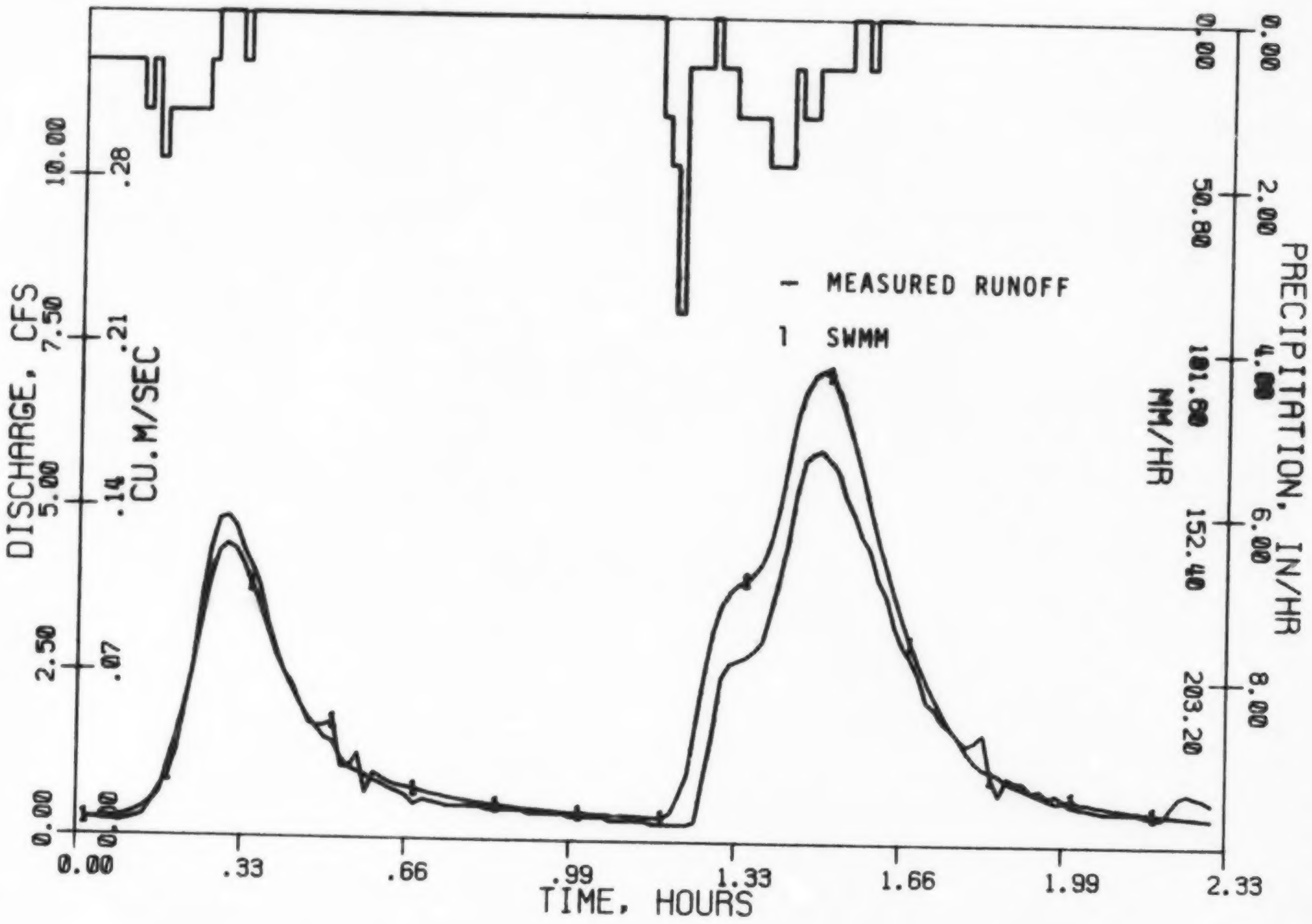

Figure 23. Oakdale storm of August 2, 1963 - EPA Storm Water Management Model

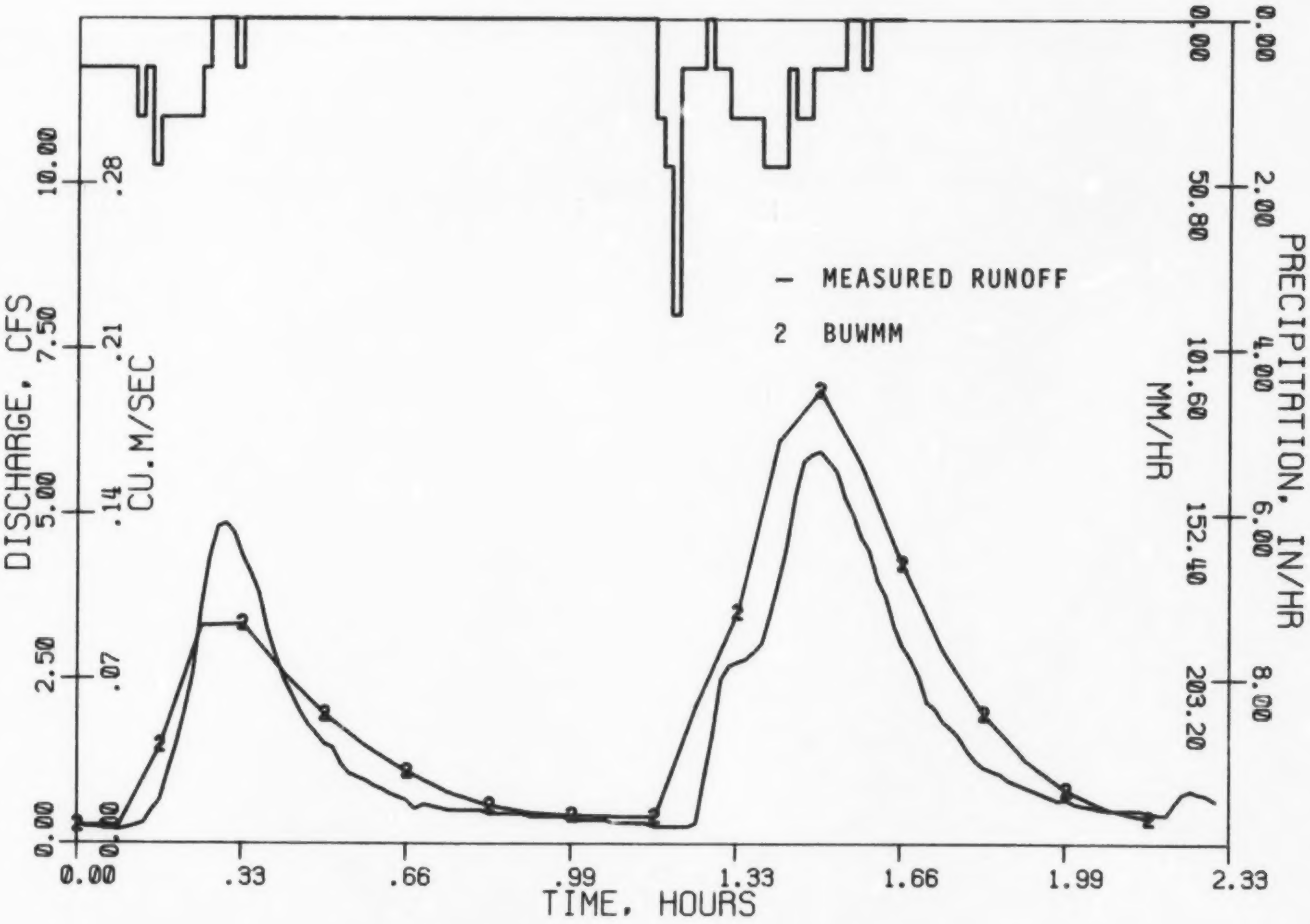

Figure 24. Oakdale storm of August 2, 1963 - Battelle Urban Wastewater Management Model

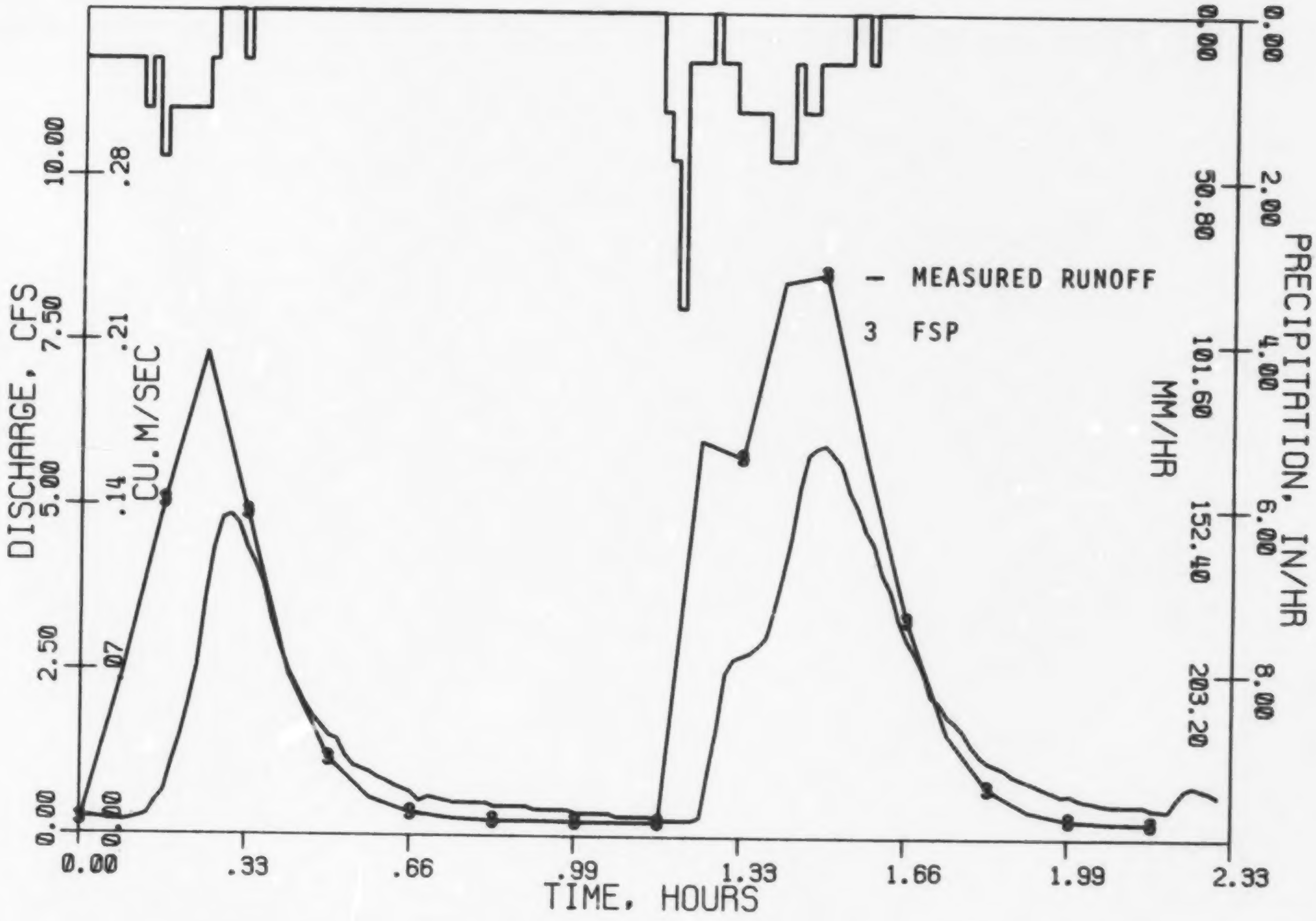

Figure 25. Oakdale storm of August 2, 1963 - Chicago Flow Simulation Program

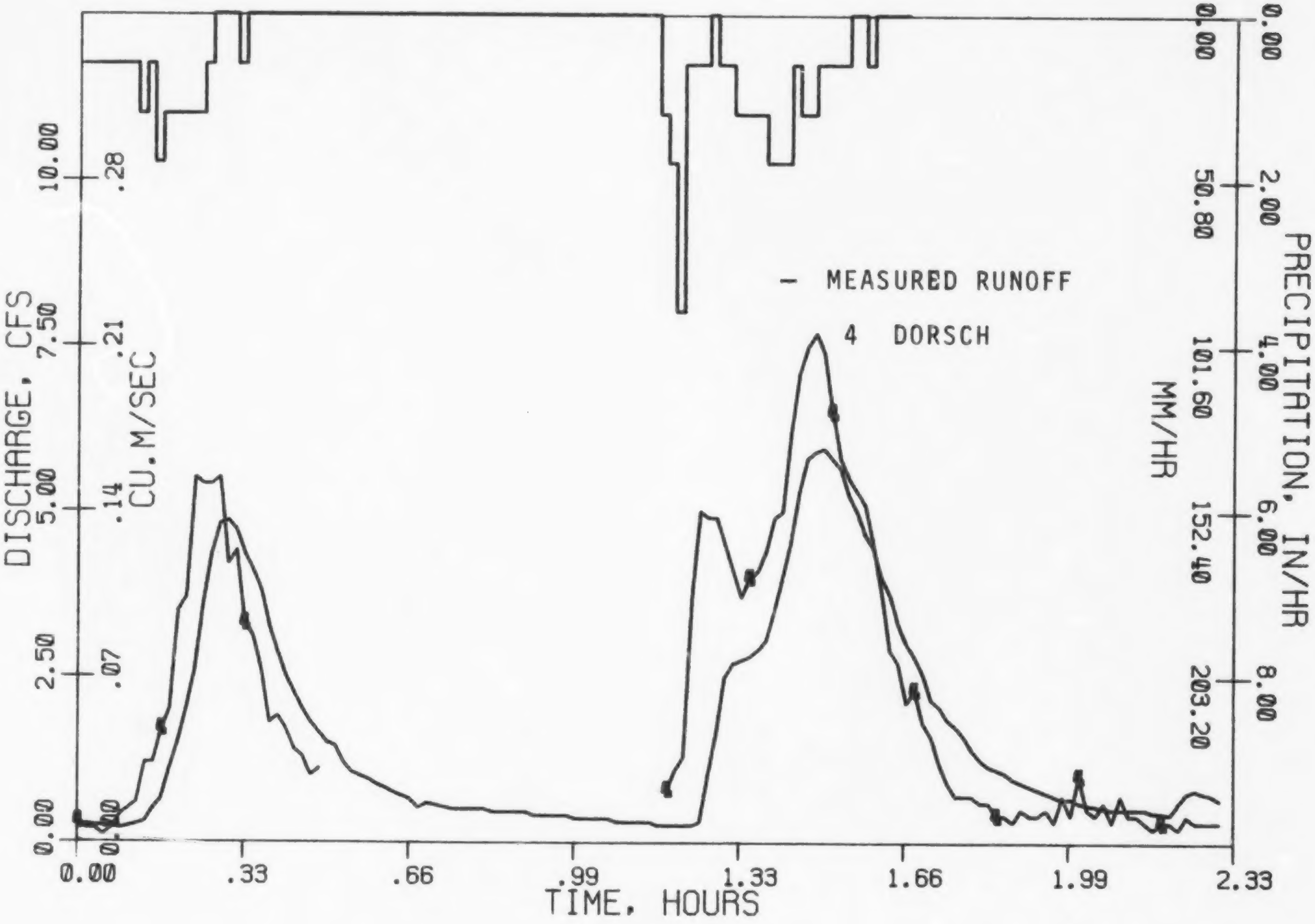

Figure 26. Oakdale storm of August 2, 1963 - Dorsch Hydrograph Volume Method

The numerical model testing indicates that equally satisfactory results can be obtained for the BUWMM, SWMM and Dorsch models for small catchments and sewerage systems where backwater and surcharging conditions do not exist or are negligible. Difficulties arise for these three models as a result of approximations in the computation of catchment moisture conditions and uncertainties with respect to initial catchment moisture conditions, particularly for long intermittent storm periods. The Dorsch model would have to be selected among these four models if backwater, downstream flow control, surcharging and pressure flow have to be simulated, but water quality does not have to be simulated. The SWMM becomes cumbersome to use and requires unnecessary computer time if only flow simulations are needed since the water quality computations cannot be suppressed and have to be performed whether needed or not.

The BUWMM has considerable flexibility in performing only portions of the modeled phenomena, and can suppress flow routing, water quality computations, realtime control optimization, and design optimization as desired by the user. The BUWMM, however, does not simulate water quality from land use, unit treatment processes, and receiving water flow and quality.

The Chicago FSP is suited primarily for the continuous simulation of large nonsewered catchments, but its general applicability may be limited due to considerable simplifications for the catchment hydrology and channel flow routing and due to the need for modifications of model coefficients internal to the computer program.

CONCLUSIONS

All of the 18 reviewed mathematical models are suitable for the simulation of storm and combined sewerage systems or for incorporation in comprehensive simulation models. Considerable differences exist, however, in the types of phenomena that are modeled and in the mathematical formulations for each phenomenon. The model reviews in a previous section summarize the objectives, advantages and limitations of each model. For some applications, models are available with considerable simplifications in their mathematical detail to reduce input data requirements, computer storage requirements, and computer running time. Some models include unnecessary approximations considering present state-of-the-art of hydrologic modeling and computer capability. Some of the simplifications, however, are based on the need for realtime control of overflows using a small process computer with slow execution times but the requirement of repeated simulations within fixed time constraints.

Tables 1 to 5 can be used to select a model based on features needed for simulating specific storm and combined sewerage system conditions. If only a few selected physical phenomena need to be modeled, then the simplest model simulating these phenomena with adequately accurate mathematical formulations should be selected. Generally, input data requirements and computer running times decrease with decreasing complexity of the model. Some models include options to suppress portions of the simulation if only selected phenomena are of interest. Although this feature is not listed in the tables, it should be considered in the model selection.

ACKNOWLEDGMENTS

The work performed for this study was conducted under contract No. 68-03-0251 of the U.S. Environmental Protection Agency with valuable guidance from Messrs. Chi-Yuan Fan (project officer), Richard Field, and Harry C. Torno. Battelle staff members contributing significantly include Mr. Larry V. Kimmel, who performed much of the model and data collection, and Mr. Stacy E. Wise and Ms. Annette S. Myhres, who performed the computer program conversions and data analyses. The cooperation of model developers and users is greatly appreciated. Dr. Paul E. Wisner of MacLaren, Ltd., Toronto, Canada, furnished the data for the Oakdale catchment in the SWMM formats.

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SHORT COURSE
APPLICATIONS OF STORMWATER MANAGEMENT MODELS

LIST OF PARTICIPANTS

Donald D. Adrian
Dept. of Civil Engineering
Univ. of Mass.
Amherst, MA 01002

Patrick A. Ahern
Dept. of Civil Engineering
University of Toronto
Toronto, Ontario Canada

Mark E. Alpert
MD Environmental Service
Tawes State Office Building
Annapolis, MD 21401

Charles App
EPA Region III
716 Raynham Rd.
Collegeville, PA 19426

K. E. Armstrong
Alberta Environment
305 Southridge
45 Ave & 106 St.
Edmonton, Alberta Canada

David Aronson
US Dept. of Int. Geol.Survey
19 Laurel Place
Bethpage, NY 11714

Laurence E. Benander
Gannett Fleming Corddry Fleming, Inc.
PO Box 1963
Harrisburg, PA 17105

Scott P. Berdine
EPA
1421 Peachtree St. NE
Atlanta, GA 30309

Bernard B. Berger
Director of Water Resources Research
Center
Univ. of Mass.
Amherst, MA 01002

Ken Black
Dept. of Public Works
Greenfield, MA 01301

Dennis D. Blair
City of Phila. Water Dept.
Room 1270, Municipal Service Bldg.
15th & JFK Blvd.
Philadelphia, PA 19107

Harry Bostian, Staff Engineer
Advanced Waste Treatment Res. Lab
National Environmental Research Center
Cincinnati, Ohio 45268

Albin Brandstetter, Manager
Environmental Resources Planning Sec.
Battelle Pacific Northwest Labs.
Richland, WA

Alvan Bruch
Independent Consultant
113 Harvard St.
Cambridge, MA 02139

Ted S. Buczek
Cleveland Reg. Sewer District
3090 Broadway Ave.
Cleveland, Ohio 44115

Theodore B. Burger
Nassau County Health Dept.
26 Coolidge Ave.
Glen Head, NY 11545

Donald Carr
R.V. Anderson Assoc.
Toronto, Ontario Canada

Carlos Carranza
Springfield College
Springfield, MA

Steven C. Chapra
GLERL/NOAA
2300 Washtenow Ave.
Ann Arbor, MI 48104

Max S. Clark
Camp Dresser & McKee, Inc.
1 Center Plaza
Boston, MA 02108

Raul Cuellar
Center for Research in Water Res.
Rt. 4, Box 189
Austin, TX 78757

K. C. Das
VA State Water Control Board
4010 W. Broad St.
Richmond, VA 23230

Francis A. DiGiano
Asst.Prof. of Civil Engineering
Univ. of Mass.
Amherst, MA

J. Obiukwu Duru
Howard Univ., Washington, D.C.
4004 Hanson Oaks Dr.
Landover Hills, MD 20784

H. Lawrence Dyer
Argonne National Laboratory
Argonne, Ill. 60439

John B. Erdmann
Mass. Div. of Water Poll. Control
PO Box 545
Westborough, MA 01581

Chi-Yuan Fan, Staff Engineer
Storm & Combined Sewer Section
Advanced Waste Treat. Res. Lab.
National Environmental Res. Center
Woodbridge Ave.
Edison, NJ 08817

Dr. T. H. Feng
Dept. of Civil Engineering
Univ. of Mass.
Amherst, MA 01002

Richard Field, Staff Engineer
Storm & Combined Sewer Section
Advanced Waste Treatment Res. Lab.
National Environmental Research Center
Woodbridge Ave.
Edison, NJ 08817

Gerald W. Foess
Curran Associates, Inc.
182 Main St.
Northampton, MA 01060

David Gaboury, Research Assistant
Dept. of Civil Engineering, Univ. of Mass.
175 Summer St., Apt. 15
Amherst, MA 01002

James A. Hagarman
Univ. City Science Center
3508 Science Center, Suite 100
Philadelphia, PA 19104

James P. Heaney
Assoc. Prof. of Environmental
Engineering Sciences
University of Florida

Howard C. Holtan
234 East 23rd Ave.
Anchorage, Alaska 99503

Wayne C. Huber
Assoc. Prof. of Environmental
Engineering Sciences
University of Florida

Thomas K. Jewell, Research Assistant
Dept. of Civil Engineering, Univ. of Mass.
RR2, 35 Harkness Rd
Amherst, MA 01002

Milton B. Kendrick
Polytech, Inc., Consulting Engineer
1836 Euclid Ave.
Cleveland, OH 44115

Alan W. Klimek
210 East Peace St.
Raleigh, NC 27611

Kenneth Klos
Brevard Engineering Co.
5750 Major Blvd.
Orlando, Fla 32805

James Kutzman
EPA
1421 Peachtree St.
Atlanta, GA 30309

E. E. Lindsey
Chemical Engineering
Univ. of Mass.
Amherst, MA 01002

Kuang-Mei Lo
Connell Assoc., Inc.
1320 S. Dixie Highway
Coral Gables, Fla. 33193

David J. Lystrom
US Geological Survey
Water Research Division
PO Box 3202
Portland, OR 97208

Dwight A. MacArthur
O'Brien & Gere Engineers, Inc.
1304 Buckley Rd.
Syracuse, NY 13201

John R. MacLennan
E. A. Hickok & Assoc.
545 Indian Mound
Wayzata, MN 55391

Peter A. Mangarella
Asst. Prof. of Civil Engineering
Univ. of Mass.
Amherst, MA 01002

Mario Marques
Federal Highway Administration
110 Lynbrook Ct.
Greenbelt, MD 20770

Richard N. Marshall
Oceanographic & Environmental Services
Raytheon Co.
P.O. Box 360
Portsmouth, RI

Jerald F. McCain
US Geological Survey
7417 S. Lamar St.
Littleton, Colo.

Robert F. McGhee
EPA, Region III
603 Allen Lane
Media, PA 19063

Murray B. McPherson, Director
ASCE Urban Water Resources Program

Walter A. Mechler
Gr. Vancouver Sewerage & Drainage Dist.
2294 W. 10th Ave.
Vancouver, BC Canada V6K 2H9

Anne N. Miller
EPA Region II
26 Federal Plaza
New York, NY 10007

Edward A. Moe
Tighe & Bond, Inc.
268 Howard St.
Ludlow, MA 01056

Michael Moss
Dept. of Natural Resources
Environmental Protection Div.
47 Trinity Ave., SW
Atlanta, GA 30334

Joanne Murphy
The Franklin Institute
20th & Cherry Sts.
Philadelphia, PA 19103

Thomas J. Murphy
Fay, Spofford & Thorndike, Inc.
11 Beacon St.
Boston, MA 02108

Rogert Nemergut
Cahn Engineers
52 Trumbull St.
New Haven, CT

George A. Nossa
US EPA Region II
26 Federal Plaza
New York, NY 10007

Glenn A. Ofcarcik
Pharmer Engineering Corp.
582 Amherst Rd.
S. Hadley, MA 01075

Gerald T. Orlob, Partner
G. T. Orlob & Associates
Orinda, CA

August B. Palmitessa
EPA Region II
26 Federal Plaza, Rm. 837
New York, NY 10007

William Paraskevas
Chem. & Biochem. Engin. Dept.
Rutgers Univ., Engineering College
New Brunswick, NJ 08903

Donald B. Partridge III
Chester Engineers
229 Goldsmith Rd
Pittsburgh, PA 15237

Richard Perna
EPA, Municipal Permits
26 Federal Plaza, Rm. 837
New York, NY 10007

Stephen E. Poole
Nashua River Program
76 Summer St., #117
Fitchbrug, MA 01420

Thomas E. Radigan
O'Brien & Gere Engineers, Inc.
1304 Buckley Rd.
Syracuse, NY 13201

Richard S. Reed
Metcalf & Eddy Engineers
Bolton Rd.
Harvard, MA 01451

J. Steve Reel
Central & Southern Florida Flood Control District
PO Box V
West Palm Beach, Fla 33402

Larry A. Roesner, Principal Engineer
Water Resources Engineers, Inc.
Walnut Creek, CA

Victor Scottron
Director, Inst. of Water Resources, CT
U-37, Univ. of Conn.
Storrs, CT 06268

Kent Scully
Elson T. Killam Assoc., Inc.
48 Essex St.
Millburn, NJ

Joe Shane
210 East Peace St.
Raleigh, NC

Robert P. Shubinski, Principal Engineer
Water Resources Engineers, Inc.
Springfield, VA

Cheryl Signs
Leonard Rice Con, Water Engin., Inc.
2785 North Speer Blvd
Denver, Colo. 80211

Joseph Skupien
Elson T. Killam Assoc., Inc.
43 Essex St.
Millburn, NJ

Richard P. Slutzah
Polycomp Sys., Inc.
188 Montague St.
Brooklyn, NY 11201

Richard D. Stalker
Area Planning Board of Palm Beach Cty.
PO Box 3643
West Palm Beach, Fla 33402

Tony Tafuri, Staff Engineer
Storm & Combined Sewer Sect.
Advanced Waste Treatment Res. Lab
National Environmental Research Center
Woodbridge Ave.
Edison, NJ 08817

John S. Tapp
EPA
1421 Peachtree St. NE
Atlanta, GA 30309

Harry Torno, Staff Engineer
Munic. Pollution Control Div. (RD67A)
Transport & Trt. Sys. Branch
EPA
Waterside Mall
Washington, DC 2046-

William P. Tully
State University of New York
College of Environm. Science & Forestry
Syracuse, NY

Frank G. Underwood
Wright, Pierce, Barnes & Wyman
25 Vaughan St
Portsmouth, NH

Sandor Vamosi
Proj. Eng. of Assoc. Eng. Services
15007 56 Ave.
Edmorton, Alberta Canada

Guillermo J. Vicens
Resource Analysis, Inc.
1033 Mass. Ave.
Cambridge, MA 02138

Jekabs P. Vittands, Project Manager
Metcalf & Eddy Consulting Engineers
Boston, MA

Vladimir Vlahovich (10CC)
United Engineers & Constructors
1401 Arch St.
Philadelphia, PA 19105

James Walsh
Springfield College
Springfield, MA

Andrew Warren
EPA
26 Federal Plaza
New York, NY 10007

Alan Wengell
Cahn Engineers
52 Trumbull St.
New Haven, CT

David Winslow
Espey-Huston & Assoc., Inc.
Austin, TX

Chin-Lien Yen
Howard University, Washington, D.C.
111 Rosemere Ave.
Silver Spring, MD 20904

Nabil Zaghoul
James F. MacLaren, Inc.
435 McNicoll Ave.
Willowdale, Ontario M2H 2R8